Submitted in Partial Satisfaction of the Requirements for

the Degree of Master of Engineering

REPAIRS TO CONCRETE PORT AND HARBOR STRUCTURES

by

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18 NOVEMBER 1991

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ABSTRACT

This report summarizes a study of repairs to concrete port and harbor structures. The study addressed the following topics:

* Characterization of the marine environment
* Causes of deterioration
* Assessment, inspection and repair methods

The study also includes an in the harbor repair project to rehabilitate pilings supporting a facility at the Alameda Naval Air Station. The project is described including lessons learned.

INTRODUCTION

The repair of concrete port and harbor structures is one element of a larger problem in the United States and world wide. That is the general decay of infrastructure as a result of age, design and construction deficiencies, and a scarcity of funds for maintenance.

"The story of our crumbling infrastructure is in many ways the story of our crumbling concrete.....despite its seeming indestructibility, concrete is susceptible to the elements and to a variety of particularly modern punishments. This, along with scarce funding for maintenance, has resulted in a literally crumbling infrastructure that will take $1 trillion to repair". [59]

This report examines the concrete port and harbor structure portion of that crumbling infrastructure in an effort to understand the ways in which deterioration occurs and how it can be prevented, arrested or repaired.

Concrete has many properties which makes it suitable and desirable for use in the marine environment. Quality concrete can be produced which has long-term strength gain, low permeability, self healing on cracking and self protection.

Concrete can be cast in place or prefabricated and installed either onshore or offshore.

Concrete has been used successfully in the marine environment for hundreds of years. One the most prominent civil engineering landmarks in the world is the Eddystone lighthouse which was constructed by the British engineer, John Smeaton, in 1756. The lighthouse is situated on the west outlet of the English Channel and was constructed before the advent of portland cement. Smeaton invented a hydraulic (water-resisting) lime by calcining limestone-clay mixtures. Smeaton's invention proved to be the precursory technology leading to the invention of portland cement in 1824. [18]

There are many well preserved and serviceable concrete structures such as the Eddystone lighthouse as well as numerous port and harbor structures located throughout the world which give testimony to the durability of good concrete in a hostile marine environment. However, there also are examples of concrete structures which are suffering from premature deterioration and failure indicating that skill and knowledge is required in the design, construction and maintenance of concrete structures in order to ensure satisfactory long-term performance.

In the following sections of this report, we characterize the marine environment for concrete port and harbor structures, review causes of deterioration, discuss methods for assessing deteriorated structures, and review repair methods currently in use. These sections are augmented with interviews of professionals in the field of marine concrete repair.

Also included herein is a case study from a repair project engineered by the authors, which incorporated all of the elements of this study in an actual repair effort. This case study afforded the rare opportunity to practice in the field what is at times too easy do discuss in the classroom.

As one Civil Engineering Professor at Northwestern University has said, "The problem with our infrastructure is not really the knowledge, the problem is getting the knowledge out of the classroom and into practice" [59].

The authors gratefully acknowledge the divers of the U.S. Navy's Shore Intermediate Maintenance Activity at the Naval Air Station, Alameda for their tireless efforts during the repair project case study. Also critical to the completion of the project were the Seabees of Construction Battalion Unit 416 who constructed

the forms for the concrete encasement, and provided invaluable assistance in the procurement of project materials. At U. C. Berkeley, Professor R. G. Bea provided necessary direction and many resources to keep our independent study on track.

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SECTION I

CHARACTERIZATION OF THE MARINE ENVIRONMENT FOR CONCRETE PORT AND HARBOR STRUCTURES

by

Jim Schofield

A. Introduction

The design, placement and maintenance of concrete for port and harbor structures includes most of the same considerations that apply to concrete on land. Design, placement, and maintenance of concrete in the marine environment, however, also requires the consideration of several additional factors. This section characterizes those additional elements of the environment which make marine concrete an exceptional case. Environmental influences to be characterized include: salinity, temperature, tides, waves, marine growth, and a unique combination of all of these influences in what is known as the "splash zone".

The splash zone referred to in this report is the zone on a marine structure that exists between the high and low water "marks" on the structure. Those high and low water marks are defined not only by the action of tides, but by that of wind and waves that subject the structure to alternating wetting and drying cycles.

One of the basic tenets of engineering is that systems most often fail when they are in a state of transition. For concrete port and harbor structures, the splash zone is in a constant

state of transition between wetting and drying and is thus of greatest concern to facility engineers.

Following the characterization of the marine environment for port and harbor structures, later sections of this report will address the effects of these environmental characteristics on deterioration and repairs to concrete structures in harbors.

B. Temperature

The temperature of harbor waters varies with location and season. Figure (I.1) presents a tabulation of average surface temperature of the ocean between parallels of latitude, and may be used as a general indicator of conditions that may be encountered on a specific job site, but more detailed local information will be necessary for a given project.

The effects of temperature on repairs to concrete port and harbor structures are five fold. They include effects on corrosion rates, the freeze thaw cycle, marine growth, curing of repair materials, and on diver performance during inspection and repair. Each of these is discussed below.

1. Corrosion Rate

In brief terms, corrosion rates are accelerated by an increase in ambient water temperature. In the Arctic, cold water temperatures slow some corrosion and deterioration processes to near zero, whereas in the tropics, extremely rapid rates of corrosion have been observed. Though nothing can be done to mitigate the ambient water temperature, it is important to recognize the characteristic temperature of water on a given project site and to provide for its effects in a repair procedure design.

2. Freeze - Thaw Cycle

In the same way that road surfaces experience seasonal cracking due to freezing and thawing of water in the pores of the asphalt or concrete, port and harbor structures experience the same phenomenon in climactic zones where freezing temperatures occur. During non-frozen months, typical permeability of the concrete will allow a certain amount of water into the pores of the concrete. When that water freezes, it expands, placing the concrete in tension, and causing local cracks. Mitigation of this action is best achieved by use of low permeability concrete to prevent the entry of significant amounts of water into the

concrete pores. This is discussed in greater detail in section II of this report.

3. Marine Growth

The development of and effects of marine growth on concrete port and harbor structures is discussed in detail below, but in terms of temperature, there is a simple relationship between the two. Marine growth rate is most often accelerated by an increase in temperature. Mitigation of this factor is best achieved by inclusion of additional cleaning of surfaces to be repaired just prior to placement of new material. This insures that the bond between materials will be the best possible.

4. Curing of Repair Materials

As with other factors above, rates of curing of repair materials are typically accelerated by higher ambient water temperature. In the case of cementitious materials, this may be detrimental to the process because for voluminous pours of new concrete, it is desirable to carry away as much of the heat of hydration as possible to prevent later shrinkage cracking. On the other end of the spectrum, epoxy based repair materials may cure too slowly or not at all if water temperature is too low. In either

case it is important for the engineer to recognize the relationships between the selected repair material and ambient temperature.

5. Diver Endurance

In the performance of any diving operation, endurance of divers is limited by the ambient water temperature. If the quality of the diver's equipment is such that they are well protected from the cold water (e.g. a heated drysuit), then their productivity is reduced by a loss of mobility in the cumbersome equipment. If the diver is directly exposed to even relatively warm water, heat transfer away from his body will limit his endurance.

There are several ways to mitigate the effect of cold water on diver endurance, and its resultant effect on productivity. First is to minimize the complexity and number of diving tasks. Diving is inherently more manpower intensive than other repair activities and this is worsened by low diver endurance in cold water. Second is to establish a good balance between thermally protective clothing and diver mobility. Third is to insure that where the pace of operations is driven by factors other than the cost of diving operations, that adequate numbers of divers

are available to counterbalance their relatively short endurance in cold water.

C. Salinity

Salinity of sea water is one of the greatest sources of the oceans ability to consume man made structures. It is defined as "the total amount of solid material in grams contained in one kilogram of seawater when all the carbonate has been converted to oxide, the bromine and iodine replaced by chlorine, and all organic matter completely oxidized". [44]

Salinity is typically stated in parts per thousand (o/oo), and typical values world wide are shown in figure (I.2). It is important to note that though we most often think of salinity as salt (sodium chloride) in sea water, that there are other elements which contribute to the total salinity of seawater. A breakdown of the major constituents of seawater is tabulated in figure (I.3) . As outlined in section II of this report, several of the constituents of seawater play key roles in the deterioration of concrete port and harbor structures.

D. Tides

"The longest waves known in the ocean are those associated with the tidal movement, which manifests itself on the coast by the rhythmic rise and fall of the water and particularly in the sounds (or harbors) and narrow straits, by the regular changing of tidal currents. The wavelike character of the phenomenon is readily recognized by means of an automatic tide gauge, which records the actual sea level of a smooth curve with alternating maxima and minima." [44].

Tides are caused by the gravitational relationships between the moon, the sun, the earth, and to a lesser extent the gravitational pull of other celestial bodies. Suffice it to say that depending on the time of year, position on the earth, and the current relative positions of the bodies mentioned, the tides will vary in some predictable and directly measurable fashion. Figure (I.4) shows the three major types of tide cycles described below:

Diurnal - characterized by one high tide and one low tide every day

Semi Diurnal - characterized by two high tides and two low tides every day

Mixed - characterized by two high tides and two low tides every day, where the magnitude of the tides are not equal on each high and low (as in San Francisco).

Terms related to the range of tides which are important to the design engineer because they affect the size of the splash zone on a structure are as follows:

Mean Higher High Water - In a semi-diurnal tide cycle, the average height of the higher of the two high tides.

Mean Lower High Water - In a semi-diurnal tide cycle, the average height of the lower of the two high tides.

Mean Higher low Water - In a semi-diurnal tide cycle, the average height of the higher of the two low tides.

Mean Lower Low Water - In a semi-diurnal tide cycle, the average height of the lower of the two low tides.

Similarly descriptive terms apply for diurnal and other tide patterns, and the key to these terms is that they identify for the engineer the range of the tides for the purposes of defining the tidal zone on a structure, and for the purpose of optimizing repair schedules to make maximum use of lowest water levels. Tide prediction information is typically available in local tide tables, an example of which is shown in figure (I.5). Figure (I.6) shows a partial summary of tide data for U.S. Naval Activities. Note that for design and repair purposes, historical data is available for statistics like "Extreme High Water", and "Extreme Low Water" . Other than obvious design criteria, these figures are useful for construction and repair sequence planning , especially on structures which are fully below the water level at mean lower low water. If work can be timed so that difficult tasks are scheduled for these "dry" times, considerable savings in manpower may be achieved, and operations that may have required divers can be handled by surface construction forces.

For example, the case study reported in Section II of this report involved concrete encasement of reinforced concrete piles on a waterfront structure. At the lower of the two daily low tides, all but the deepest row of piles were exposed to the mudline. Some operations during these low tides were an order of

magnitude faster than when the piles were partially or fully submerged.

E. Waves

In a properly designed harbor, wave forces are not typically of a magnitude which will directly govern the design of structures within the harbor. In most cases, the design of reinforced concrete piers, wharves, and associated structures is driven by operational loads that the structure will experience, namely: mooring loads, collision loads, and cargo handling live and dead loads. It is important, however, to understand the effect of waves in a harbor on the deterioration of harbor structures, particularly the wave's ability to induce what is commonly known as "mechanical" damage. A thorough knowledge of the wave environment in a given harbor will allow for more accurate evaluation and effective correction of deterioration of the harbor structures. Appendix I provides a more detailed evaluation of the influence of waves on the repair of deteriorated concrete port and harbor structures.

F. Marine Growth

1. Introduction

Although biological growth most often damages concrete port and harbor structures which are constructed of unsound, soft concrete, the presence of biofouling on concrete structures to be repaired is important to both the inspection process and to the repair process. In the inspection process, biofouling can limit the inspector's ability to see the structure, and in many cases can mask or obscure damage. This is discussed in greater detail in section III of this report. For concrete repairs underwater, marine growth can also greatly reduce the bond strength of new cementitious and epoxy materials,and thus must be cleaned off prior to repair. Section IV provides additional details on surface preparation.

2. Destructive Organisms

As mentioned above, biological growth is typically only destructive of concrete port and harbor structures when those structures are constructed of poor quality, soft concrete. In that case, however, there are two types of organisms of note in this form of deterioration. First are boring clams and mollusks, and second are sulfate producing bacteria that can exist anaerobically at the mudline interface. Both types of organisms are discussed in greater detail in Section II of this report.

3. Growth Rate

The rate at which algae and other marine organisms will grow and inhibit **bonding** is dependent on several factors as described below.

Depth- With increasing depth, there is a decrease in temperature, light penetration, and availability of food, and an increase in pressure. The first three effects have a marked effect in limiting the metabolism and rate of growth of marine organisms, and thus are responsible for the general thinning trend in marine growth with depth. For the purposes of "last minute" cleaning prior to placement of repair materials, algae are of primary concern, and because they depend on light for metabolism, algal cover and growth rate decrease with depth.

Temperature - A rise in temperature usually increases the growth rate of a community. A figure of doubled growth rate per 10 degrees Celsius provides a rough guide. For pre-repair cleaning, suffice it to say that in warmer water, the time between cleaning and placement of repair materials is more critical than in colder water.

Seasonal Effects - For pre-repair cleaning, seasonal effects are reduced to the simple temperature effect described above.

Salinity - A reduction in the quantity and diversity of marine growth is generally noticeable with decreasing salinity, but there are typically types of algae which flourish in a given salinity. Thus, decreasing salinity is not necessarily an aid to the surface preparation process.

G. Summary

The wide variety and range of environmental factors which impact the need for and conduct of repairs to concrete port and harbor structures make this arena a particularly challenging one. Current techniques for environmental data gathering in combination with historical records must play key roles in the repair of concrete port and harbor structures to ensure that adequate consideration is given to each of the environmental factors mentioned above when selecting and implementing a repair procedure. It is the task of the engineers formulating repair procedures to insure that shortfalls in the consideration of environmental factors during original design are not repeated in the repair process.

AVERAGE SURFACE TEMPERATURE OF THE OCEANS BETWEEN PARALLELS OF LATITUDE

North latitude	Atlantic Ocean	Indian Ocean	Pacific Ocean	South latitude	Atlantic Ocean	Indian Ocean	Pacific Ocean
70°–60°.....	5.60	.....	.....	70°–60°.....	– 1.30	– 1.50	– 1.30
60 –50......	8.66	.....	5.74	60 –50......	1.76	1.63	5.00
50 –40......	13.16	.....	9.99	50 –40......	8.68	8.67	11.16
40 –30......	20.40	.....	18.62	40 –30......	16.90	17.00	16.98
30 –20......	24.16	26.14	23.38	30 –20......	21.20	22.53	21.53
20 –10......	25.81	27.23	26.42	20 –10......	23.16	25.85	25.11
10 – 0......	26.66	27.88	27.20	10 – 0......	25.18	27.41	26.01

FIGURE I.1 Representative Ocean Surface Temperatures For Various Latitudes and Ocean Basins [44]

AVERAGE VALUES OF SALINITY, S, EVAPORATION, E, AND PRECIPITATION, P, AND THE DIFFERENCE, $E - P$, FOR EVERY FIFTH PARALLEL OF LATITUDE BETWEEN 40°N AND 50°S
(After Wust)

Latitude	Atlantic Ocean				Indian Ocean				Pacific Ocean				All Oceans			
	S (‰)	E (cm/yr)	P (cm/yr)	$E - P$ (cm/yr)	S (‰)	E (cm/yr)	P (cm/yr)	$E - P$ (cm/yr)	S (‰)	E (cm/yr)	P (cm/yr)	$E - P$ (cm/yr)	S (‰)	E (cm/yr)	P (cm/yr)	$E - P$ (cm/yr)
40°N	35 80	94	76	18					33 64	94	93	1	34 54	94	93	1
35	36 46	107	64	43					34 10	106	79	27	35 05	106	79	27
30	36 79	121	54	67					34 77	116	65	51	35 56	120	65	55
25	36 87	140	42	98					35 00	127	55	72	35 79	129	55	74
20	36 47	149	40	110	(35 05)	(125)	(74)	(51)	34 85	130	62	68	35 44	133	65	68
15	35 92	145	62	83	(35 07)	(125)	(73)	(52)	34 67	128	82	46	35.09	130	82	48
10	35 62	132	101	31	(34 92)	(125)	(88)	(37)	34 29	123	127	− 4	34 72	129	127	2
5	34 98	105	144	−39	(34 82)	(125)	(107)	(18)	34 29	102	(177	−75)	34 54	110	177	−67
0	35 67	116	96	20	35 14	125	131	− 6	34 85	116	98	18	35 08	119	102	17
5°S	35 77	141	42	99	34 93	121	167	−46	35 11	131	91	40	35 20	124	91	33
10	36 45	143	22	121	34 57	99	156	−57	35 38	131	96	35	35 34	130	96	34
15	36 79	138	19	119	34 75	121	83	38	35 57	125	85	40	35 54	134	85	49
20	36 54	132	30	102	35 15	143	59	84	35 70	121	70	51	35 69	134	70	64
25	36.20	124	40	84	35 45	145	46	99	35 62	116	61	55	35 69	124	62	62
30	35 72	116	45	71	35 89	134	58	76	35 40	110	64	46	35 62	111	64	47
35	35 35	99	55	44	35 60	121	60	61	35 00	97	64	33	35 32	99	64	35
40	34 65	81	72	9	35 10	83	73	10	34 61	81	84	− 3	34 79	81	84	− 3
45	34 19	64	73	− 9	34 2[illegible]	64	79	−15	34 32	64	85	−21	34 14	64	85	−21
50	33 94	43	72	−29	33 87	43	79	−36	34 16	43	84	−41	33 99	43	84	−41

FIGURE I.2 Representative Values of Salinity of Seawater For Various Latitudes and Ocean Basins [44]

MAJOR CONSTITUENTS OF SEA WATER
($Cl = 19.00\,‰$, $\rho_{20} = 1.0243$)

Ion	‰	Cl-ratio, g per unit Cl	Equivalent per kg of sea water	mg-atoms per liter	Chlorosity factor, mg-atoms per unit *Cl*	Authority
Chloride, Cl^-	18.9799	0.99894	0.5353	548.30	28.17_3	Dittmar (1884), Jacobsen and Knudsen (194
Sulphate, $SO_4^{=}$	2.6486	0.1394	0.0551	(SO_4-S) 38.24	1.45_1	Thompson, Johnston, and Wirth (1931)
Bicarbonate, HCO_3^-	0.1397	0.00735[a]	0.0023	(HCO_3-C) 2.34	0.12_0	Revelle (1936)
Bromide, Br^-	0.0646	0.00340	0.0008	0.83	0.042_6	Dittmar (1884)
Fluoride, F^-	0.0013	0.00007	0.0001	0.07	0.003_6	Thompson and Taylor (1933)
Boric acid,[c] H_3BO_3	0.0260	0.00137[b]	[c]	(H_3BO_3-B) 0.43	0.022_1	Harding and Moberg (1934), Igelsrud, Thompson, and Zwicker (1938)
Total			0.5936			
Sodium,[d] Na^+	10.5561	0.5556	0.4590	470.15	24.15_8	By difference, and Robinson and Knapman (1941)
Magnesium, Mg^{++}	1.2720	0.06695	0.1046	53.57	2.75_3	Thompson and Wright (1930)
Calcium, Ca^{++}	0.4001	0.02106	0.0200	10.24	0.526_2	Kirk and Moberg (1933); Thompson and Wright (1930)
Potassium, K^+	0.3800	0.02000	0.0097	9.96	0.511_8	Thompson and Robinson (1932)
Strontium, Sr^{++}	0.0133	0.00070	0.0003	0.15	0.007_7	Webb (1938)
Total			0.5936			

Total dissolved solids = 34.4816 ‰
Sum of constituents (HCO_3^- as $O^=$, and Br^- as Cl^-) = 34.324 ‰
Salinity ($S\,‰ = 0.030 + 1.805\,Cl\,‰$) = 34.325 ‰
[a] Ratio for millival/kg = 0.1205
[b] Ratio for boron/Cl = 0.000240
[c] Boric acid undissociated
[d] Sodium calculated by difference in sums of equivalents

FIGURE I.3 Major Constituents of Seawater [44]

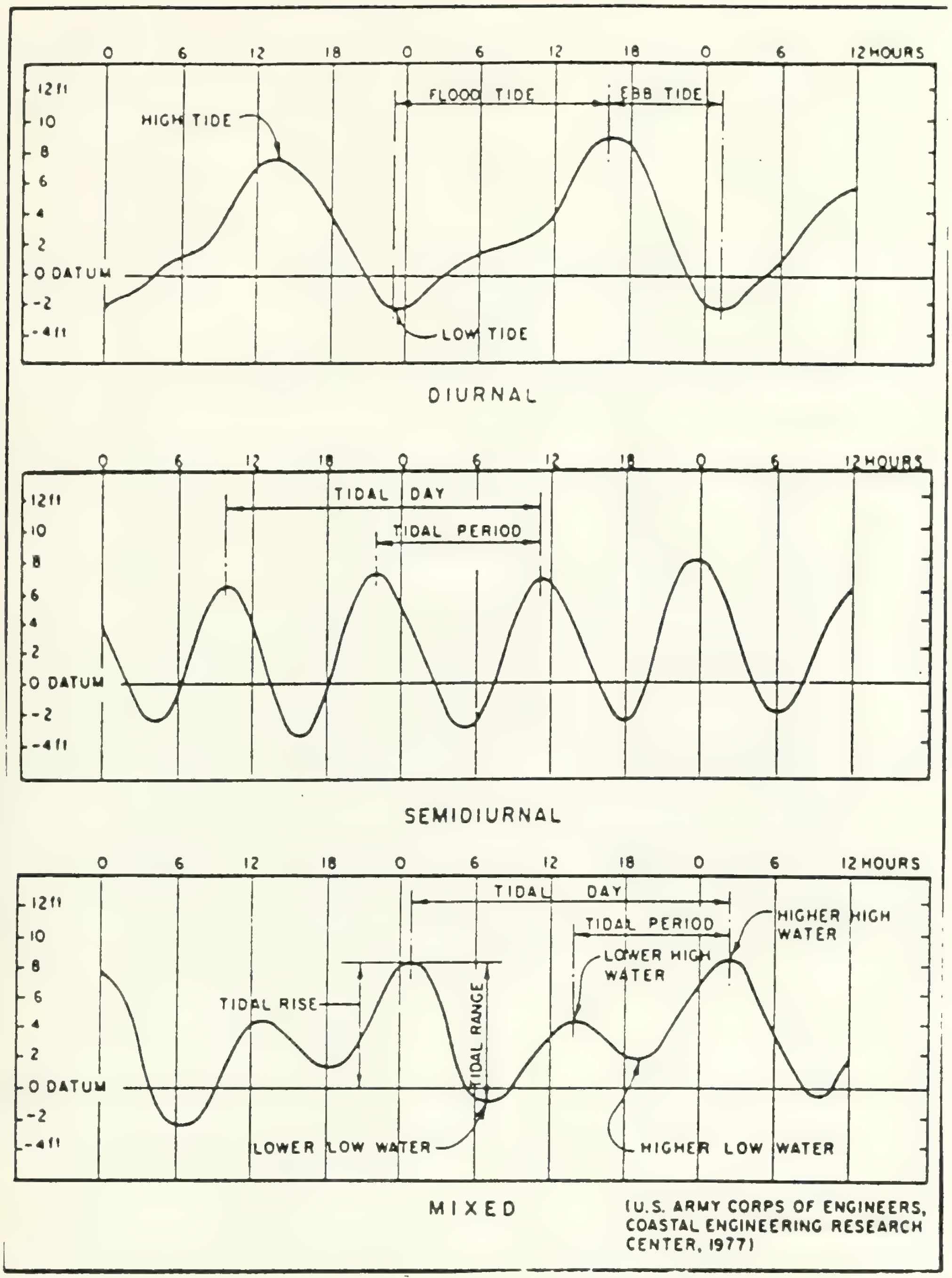


FIGURE I.4 Types of Tides and Tide Nomenclature [27]

SANDY HOOK, N.J.

TIMES AND HEIGHTS OF HIGH AND LOW WATERS

JULY

DAY	TIME h.m.	HEIGHT ft.	m.	DAY	TIME h.m.	HEIGHT ft.	m.
1	0354	-0.5	-0.2	16	0429	0.0	0.0
TU	0955	4.6	1.4	M	1037	4.4	1.3
	1600	0.1	0.0		1637	0.6	0.2
	2211	5.4	1.6		2240	4.7	1.4
2	0440	-0.4	-0.1	17	0509	0.2	0.1
W	1048	4.7	1.4	TH	1122	4.4	1.3
	1650	0.2	0.1		1719	0.8	0.2
	2305	5.3	1.6		2322	4.5	1.4
3	0529	-0.3	-0.1	18	0549	0.4	0.1
TH	1143	4.8	1.5	F	1206	4.3	1.3
	[illegible]	0.4	0.1		1807	1.0	0.3
4	0001	5.0	1.5	19	0004	4.3	1.3
F	0623	[illegible]	-0.1	SA	0634	0.6	0.2
	1239	4.9	1.5		1251	4.3	1.3
	1859	0.4	0.1		1903	1.1	0.3
5	0058	4.8	1.5	20	0053	4.1	1.2
SA	0724	-0.1	0.0	SU	0726	0.7	0.2
	1336	5.0	1.5		1337	4.4	1.3
	2007	0.4	0.1		2003	1.0	0.3
6	0158	4.6	1.4	21	0142	3.9	1.2
SU	0824	-0.1	0.0	M	0819	0.7	0.2
	1438	5.1	1.6		1430	4.5	1.4
	2111	0.2	0.1		2102	0.9	0.3
7	0303	4.5	1.4	22	0239	3.9	1.2
M	0922	-0.2	-0.1	TU	0912	0.7	0.2
	1540	5.3	1.6		1527	4.6	1.4
	2212	0.0	0.0		2154	0.7	0.2
8	0409	4.5	1.4	23	0345	3.9	1.2
TU	1017	-0.2	-0.1	W	1002	0.5	0.2
	1641	5.5	1.7		1622	4.9	1.5
	2307	-0.1	0.0		2246	0.5	0.2
9	0510	4.6	1.4	24	0444	4.0	1.2
W	1110	-0.2	-0.1	TH	1051	0.4	0.1
	1734	5.7	1.7		1713	5.2	1.6
					2336	0.2	0.1
10	0001	-0.3	-0.1	25	0536	4.3	1.3
TH	0603	4.7	1.4	F	1140	0.2	0.1
	1203	-0.2	-0.1		1803	5.5	1.7
	1824	5.8	1.8				
11	0055	-0.4	-0.1	26	0027	-0.1	0.0
F	0652	4.7	1.4	SA	0626	4.5	1.4
	1254	-0.2	-0.1		1230	0.1	0.0
	1910	5.8	1.8		1848	5.8	1.8
12	0143	-0.4	-0.1	27	0117	-0.4	-0.1
SA	0738	4.8	1.5	SU	0712	4.8	1.5
	1343	-0.1	0.0		1321	-0.1	0.0
	1952	5.7	1.7		1932	5.9	1.8
13	0229	-0.4	-0.1	28	0203	-0.6	-0.2
SU	0824	4.7	1.4	M	0758	4.9	1.5
	1429	0.0	0.0		1412	-0.2	-0.1
	2035	5.5	1.7		2017	5.9	1.8
14	0310	-0.3	-0.1	29	0249	-0.7	-0.2
M	0909	4.6	1.4	TU	0845	5.1	1.6
	1514	0.1	0.0		1459	-0.3	-0.1
	2117	5.3	1.6		2105	5.9	1.8
15	0351	-0.2	-0.1	30	0333	-0.7	-0.2
TU	0952	4.5	1.4	W	0936	5.2	1.6
	1555	0.3	0.1		1549	-0.2	-0.1
	2159	5.0	1.5		2155	5.7	1.7
				31	0419	-0.7	-0.2
				TH	1029	5.2	1.6
					1640	-0.1	0.0
					2250	5.4	1.6

AUGUST

DAY	TIME h.m.	HEIGHT ft.	m.	DAY	TIME h.m.	HEIGHT ft.	m.
1	0506	-0.5	-0.2	16	0504	0.5	0.2
F	1122	5.2	1.6	SA	1122	4.5	1.4
	1737	0.1	0.0		1725	0.8	0.2
	2344	5.1	1.6		2325	4.3	1.3
2	0559	-0.3	-0.1	17	0541	0.7	0.2
SA	1218	5.2	1.6	SU	1204	4.5	1.4
	1839	0.3	0.1		1814	1.0	0.3
3	0041	4.8	1.5	18	0007	4.1	1.2
SU	0659	-0.1	0.0	M	0623	0.8	0.2
	1316	5.2	1.6		1251	4.5	1.4
	1948	0.3	0.1		1914	1.1	0.3
4	0140	4.6	1.4	19	0057	3.9	1.2
M	0801	0.0	0.0	TU	0723	0.9	0.3
	1416	5.2	1.6		1342	4.5	1.4
	2054	0.3	0.1		2019	1.0	0.3
5	0244	4.4	1.3	20	0155	3.8	1.2
TU	0901	0.1	0.0	W	0824	0.9	0.3
	1519	5.2	1.6		1439	4.6	1.4
	2154	0.2	0.1		2119	0.8	0.2
6	0351	4.3	1.3	21	0300	3.8	1.2
W	0959	0.1	0.0	TH	0924	0.7	0.2
	1622	5.3	1.6		1542	4.8	1.5
	2252	0.1	0.0		2214	0.5	0.2
7	0454	4.4	1.3	22	0409	4.0	1.2
TH	1054	0.1	0.0	F	1020	0.5	0.2
	1718	5.4	1.6		1642	5.1	1.6
	2345	-0.1	0.0		2305	0.2	0.1
8	0550	4.5	1.4	23	0510	4.4	1.3
F	1145	0.0	0.0	SA	1115	0.2	0.1
	1808	5.5	1.7		1734	5.5	1.7
					2357	-0.2	-0.1
9	0035	-0.2	-0.1	24	0601	4.8	1.5
SA	0636	4.7	1.4	SU	1208	0.0	0.0
	1236	0.0	0.0		1824	5.8	1.8
	1853	5.6	1.7				
10	0122	-0.2	-0.1	25	0047	-0.5	-0.2
SU	0720	4.8	1.5	M	0650	5.1	1.6
	1325	0.0	0.0		1301	-0.3	-0.1
	1933	5.5	1.7		1911	6.0	1.8
11	0204	-0.3	-0.1	26	0136	-0.7	-0.2
M	0802	4.8	1.5	TU	0738	5.4	1.6
	1410	0.1	0.0		1353	-0.5	-0.2
	2012	5.4	1.6		1957	6.1	1.9
12	0245	-0.2	-0.1	27	0224	-0.9	-0.3
TU	0841	4.8	1.5	W	0825	5.6	1.7
	1450	0.1	0.0		1443	-0.6	-0.2
	2049	5.2	1.6		2047	6.0	1.8
13	0321	-0.1	0.0	28	0310	-0.9	-0.3
W	0921	4.7	1.4	TH	0912	5.7	1.7
	1528	0.3	0.1		1534	-0.6	-0.2
	2127	5.0	1.5		2136	5.8	1.8
14	0357	0.1	0.0	29	0355	-0.8	-0.2
TH	1000	4.7	1.4	F	1005	5.7	1.7
	1607	0.5	0.2		1624	-0.4	-0.1
	2205	4.7	1.4		2229	5.5	1.7
15	0429	0.3	0.1	30	0442	-0.6	-0.2
F	1040	4.6	1.4	SA	1100	5.6	1.7
	1645	0.6	0.2		1719	-0.2	-0.1
	2242	4.5	1.4		2325	5.1	1.6
				31	0535	-0.3	-0.1
				SU	1156	5.4	1.6
					1820	0.1	0.0

SEPTEMBER

DAY	TIME h.m.	HEIGHT ft.	m.	DAY	TIME h.m.	HEIGHT ft.	m.
1	0022	4.7	1.4	16	0521	0.9	0.3
M	0633	0.0	0.0	TU	1201	4.5	1.4
	1252	5.3	1.6		1828	1.0	0.3
	1927	0.3	0.1				
2	0123	4.4	1.3	17	0018	3.8	1.2
TU	0737	0.3	0.1	W	0621	1.0	0.3
	1354	5.1	1.6		1258	4.5	1.4
	2033	0.4	0.1		1931	1.0	0.3
3	0227	4.2	1.3	18	0119	3.8	1.2
W	0842	0.4	0.1	TH	0740	1.0	0.3
	1457	5.0	1.5		1358	4.6	1.4
	2135	0.3	0.1		2045	0.8	0.2
4	0334	4.2	1.3	19	0227	3.9	1.2
TH	0942	0.4	0.1	F	0854	0.9	0.3
	1601	5.0	1.5		1505	4.7	1.4
	2233	0.2	0.1		2142	0.5	0.2
5	0436	4.3	1.3	20	0337	4.1	1.2
F	1038	0.3	0.1	SA	0954	0.5	0.2
	1659	5.1	1.6		1609	5.0	1.5
	2323	0.1	0.0		2236	0.1	0.0
6	0531	4.5	1.4	21	0441	4.5	1.4
SA	1129	0.2	0.1	SU	1051	0.2	0.1
	1747	5.2	1.6		1708	5.4	1.6
					2328	-0.3	-0.1
7	0011	0.0	0.0	22	0537	5.0	1.5
SU	0616	4.7	1.4	M	1147	-0.2	-0.1
	1216	0.2	0.1		1800	5.7	1.7
	1832	5.3	1.6				
8	0053	-0.1	0.0	23	0018	-0.6	-0.2
M	0657	4.9	1.5	TU	0627	5.5	1.7
	1304	0.1	0.0		1242	-0.5	-0.2
	1910	5.3	1.6		1850	6.0	1.8
9	0136	-0.1	0.0	24	0109	-0.9	-0.3
TU	0735	5.0	1.5	W	0715	5.9	1.8
	1346	0.1	0.0		1334	-0.8	-0.2
	1946	5.2	1.6		1938	6.0	1.8
10	0214	-0.1	0.0	25	0157	-1.1	-0.3
W	0811	5.0	1.5	TH	0801	6.1	1.9
	1425	0.1	0.0		1425	-0.9	-0.3
	2022	5.1	1.6		2027	5.9	1.8
11	0249	0.0	0.0	26	0243	-1.1	-0.3
TH	0848	5.0	1.5	F	0850	6.1	1.9
	1504	0.2	0.1		1516	-0.9	-0.3
	2056	4.9	1.5		2116	5.7	1.7
12	0323	0.1	0.0	27	0331	-0.9	-0.3
F	0924	4.9	1.5	SA	0941	6.0	1.8
	1539	0.3	0.1		1607	-0.7	-0.2
	2132	4.6	1.4		2209	5.3	1.6
13	0353	0.3	0.1	28	0419	-0.6	-0.2
SA	1000	4.8	1.5	SU	1034	5.8	1.8
	1613	0.5	0.2		1700	-0.4	-0.1
	2208	4.4	1.3		2306	4.9	1.5
14	0421	0.5	0.2	29	0509	-0.2	-0.1
SU	1039	4.7	1.4	M	1130	5.5	1.7
	1650	0.7	0.2		1759	0.0	0.0
	2245	4.2	1.3				
15	0450	0.7	0.2	30	0003	4.6	1.4
M	1119	4.6	1.4	TU	0608	0.2	0.1
	1732	0.8	0.2		1228	5.2	1.6
	2330	4.0	1.2		1904	0.2	0.1

TIME MERIDIAN 75° W. 0000 IS MIDNIGHT. 1200 IS NOON.
HEIGHTS ARE REFERRED TO MEAN LOW WATER WHICH IS THE CHART DATUM OF SOUNDINGS.

FIGURE I.5 Typical Tide Table [60]

Tide Data for U.S. Naval Activities

Atlantic Naval Activities										
Location	Tide Station	Latitude/ Longitude	Extreme High Water Elev. (ft)	Extreme High Water Date	MHW (ft)	MLW (ft)	Extreme Low Water Elev. (ft)	Extreme Low Water Date	Difference NGVD–Gage Dat. (ft)	Observation Period
Providence, RI...........	State Pier No. 1	41 48.4 71 24.1	17.7	21 Sep '38	4.60	0.0	-3.4	5 Jan '59	1.85 (MLW Gage)	1939–46 1957–61
Salem, MA....	Salem	42 31.1 70 53.2	14.0	(Est.)	8.80	0.0	-3.5	(Est.)	....	27 May- 15 Jun '67 1-31 Aug '67 1-15 Sep '67
Savannah, GA...........	Savannah River	32 04.8 81 04.9	12.0	(Est.)	7.50	0.0	-4.5	(Est.)	3.37 (MLW Gage)	1934–35
Solomon, MD...........	....	....	6.5	....	1.2	0.0	-3.0	....	MLW	....
Thoms Cove, MD...........	....	....	....	....	1.1	0.0	-4.5	....	MLW	....
Wilmington, NC...........	Cape Fear River	34 13.6 77 57.2	8.2	15 Oct '54	4.20	0.0	-1.7	3 Feb '40	1.52 (MLW Gage)	1969–73
Bermuda Bio. Station......	Georges	32 22.2 64 41.8	4.7	22 Nov '61	2.40	0.0	-1.9	15 Mar '45 30 Apr '46 1 May '46	....	1945–59
Canal Zone, Panama.......	Cristobol Tide Sta.	9 21.2 79 54.8	1.4	....	0.32	-0.38	-1.2	....	....	1949–54
Grondal, Greenland....	....	....	....	....	7.5	1.0	....	....	MLWS	....
Narsarssuak, Greenland....	....	....	....	....	8.6	2.0	....	....	1.0 Ft Below MLWS	....
Reykjavik, Iceland......	....	....	....	....	11.4	2.2	....	....	0.5 Ft Below MLWS	....
Trinidad, B.W.I........	Carenage Bay	10 41.1 61 36.5	2.3	....	1.0	-1.0	-2.5	....	....	1950–51

NOTE: The datum for the Atlantic and Gulf coasts is currently being changed from MLW to MLLW. This change was not completed at the time of publication. Refer to current National Oceanic and Atmospheric Administration (NOAA) tide tables for tide data referenced to MLLW datum.

Pacific Naval Activities										
Location	Tide Station	Latitude/ Longitude	Extreme High Water Elev. (ft)	Extreme High Water Date	MHHW (ft)	MLLW (ft)	Extreme Low Water Elev. (ft)	Extreme Low Water Date	Difference NGVD–Gage Dat. (ft)	Observation Period
Alameda, CA...........	S.F. Bay Naval Air Station	37 46.5 122 17.9	9.0	18 Jan '73	6.40	0.00	-2.2	12 Jun '68	3.03 (MLW Gage) "Above Lower Low Water Datum"	1941–59
Antioch, CA...........	San Joaquin River	38 01.2 121 48.9	7.0	(Est.)	3.71	0.00	-2.0	(Est.)	1.02 (MLW Gage)	1977–79*
Astoria, OR...........	Tongue Pt., Columbia River	46 12.5 123 46.0	12.1	17 Dec '33	8.30	0.0	-2.8	16 Jan '30	3.05 (MLW Gage)	1941–59
Bremberton, WA...........	Puget Sound	47 33.5 122 38.0	14.7	22–24 Dec '40	11.70	0.00	-4.5	30 Nov '36	....	1935–42
Hunters Point, CA....	S.F. Bay	37 43.8 122 21.4	9	(Est.)	6.60	0.0	-2.5	(Est.)	3.09 (MLW Gage)	1974–76*
Long Beach, CA...........	L.A. Outer Harbor	33 43.2 118 15.3	7.8	8 Jun '74	5.40	0.0	-2.6	26 Dec '32 11 Dec '33	2.72 (MLW Gage)	1941–59
Mare Island Strait, CA...	Bridge Over Mare Island	38 05.2 122 18.6	8.5	(Est.)	5.71	0.0	-2	(Est.)	2.50 (MLW Gage)	1977–78*
Port Hueneme, CA...........	Port Hueneme	34 08.9 119 12.2	7.6	4 Feb '58	5.40	0.0	-2.4	7 Jan '51	2.83 (MLW Gage)	1941–59
Port of Chicago, CA...........	Refugio Landing, San Pablo Bay	38 01.4 122 17.5	8.5	(Est.)	5.64	0.0	-2.5	(Est.)	2.51 (MLW Gage)	1976–77*

*Reduced to mean (1941-59)

FIGURE I.6 Tide Data For U.S. Naval Activities [27]

SECTION II

CAUSES OF DETERIORATION OF CONCRETE IN PORT AND HARBOR STRUCTURES

by

Max Rodgers

A. Introduction

Deterioration of concrete port and harbor structures may be the result of numerous phenomenon. The deterioration may be the result of physical, chemical, or biological attack or it may be the result of poor design and/or construction. Figure II.1 is an illustration of some of the processes responsible for the deterioration of reinforced concrete exposed to the marine environment.

Almost all of the deteriorating mechanisms in concrete port and harbor structures have in common that the deterioration involves expansion phenomena and that moisture is required for the deterioration to proceed. Since moisture is a characteristic feature of exposure of marine concrete construction, this environment is especially harsh on concrete port and harbor structures. In order to function in this harsh environment, concrete port and harbor structures must be designed, constructed and maintained with particular care and with practical technology in order to realize the longevity that is expected of them.

Deterioration of concrete port and harbor structures may be initiated by a single mechanism and continued by additional

deteriorating mechanisms. For instance, some type of mechanical damage may initiate a crack in a concrete structure into which the saltwater may intrude and begin chemical deterioration due to corrosion of the reinforcing steel. The crack may go unnoticed due to the growth of marine biofouling organisms and the corrosion continues. If the concrete port or harbor structure is located in a temperate climate, freezing and thawing may aggravate the cracking during the winter season. Furthermore, the corrosion may be accelerated due to poor construction practices and may further be complicated by the design deficiency of inadequate cover of concrete over the reinforcing steel leading to spalling of the concrete and continued deterioration of the structure.

For concrete port and harbor structures located in a warm climate, the heat itself may be a deteriorating mechanism. Heat is a driving energy source which accelerates both the onset and the progress of the deteriorating mechanisms. The classical chemistry law which ties heat and the rate of chemical reactions together states that for each increase of ten degrees Celsius in temperature, the rate of chemical reactions is doubled. The elevated temperature also has the effect of drying out the concrete, thus increasing its porosity and the potential availability of chlorides and oxygen at the reinforcing steel.

The impact of heat upon the rates of deteriorating mechanism has only recently been realized. Figure II.2 illustrates the deteriorating effect that increased temperature may have on concrete structures exposed to the marine environment.

The composite action of the deteriorating effects of the marine environment is present daily and into this harsh environment concrete port and harbor structures are constructed and expected to function to their fullest potential. The forces that are at work in the marine environment virtually assure that deterioration of concrete port and harbor structures will occur but with proper engineering, construction and maintenance, concrete port and harbor structures will continue to perform up to the expectations placed upon them. This fact is evidenced by concrete structures which have been performing satisfactorily in the marine environment for over eighty-five years.

B. Mechanical Deterioration

Severe damage to concrete port and harbor structures can occur due to various mechanical mechanisms. This damage may be the result of neglect, ignorance, or incompetency. The mechanical deterioration may be the result of man, nature or a combination of the two.

Another problem inherent to concrete port and harbor structures is that mechanical damage to the structure may be located either underwater or in a location which is not readily observable for inspection or accessible for repair.

1. Overloading

Overloading of concrete port and harbor structures can lead to deterioration. Once constructed, concrete port and harbor facilities are frequently utilized by individuals, organizations or companies that have limited, if any knowledge of the actual design capacity of the structure. During the daily utilization of the structure many forces are at work acting on a port or harbor structure which may lead to deterioration due to overloading.

Deterioration of reinforced concrete piles due to axial overload can be a cause of eventual failure of the pile and possibly the entire port or harbor structure. Overloading can be in the form of superimposed loads, both "dead" loads and "live" loads, exceeding the bearing capacity of the pile. Figure II.3 is an illustration of deterioration of a reinforced concrete pile due to overloading.

A commom cause of concrete pile deterioration is overloading at the time of driving. Hairline cracks occur in the piles which are difficult to see, especially if the pile is driven under water. As marine growth covers the pile, the cracks in the piles become extremely difficult to detect and left undetected will continue to deteriorate.

2. Impacts

Another mechanically deteriorating factor which is imposed upon concrete port and harbor structures is impact loading. Just as with overloading, impact loads may be the result of misuse of the structure, abuses by mother nature or a combination of the two.

Ships, barges and boats colliding with concrete port and harbor structures can be extremely detrimental to the structures. The

primary purpose of the majority of port and harbor structures is to provide access to shipping and therefore, the structures are in constant danger of sustaining damage due to collisions with ships, barges and boats.

The attachment of fendering devices to concrete port and harbor structures help to protect the structures from anticipated impacts, however, frequently the collisions which occur far exceed the anticipated force used to design the fendering system. When this happens, the structure must take the stress of the impact load and may suffer damage as a result.

Ships, barges and boats attempting to berth alongside concrete port and harbor structures are sometimes forced to impact the structures due to the forces of wind, waves and currents. Even though ports and harbors are designed to restrict the detrimental effects of waves and currents, these forces frequently exist and can be rather substantial.

The predominate force of nature which appears to have the greatest influence with regards to imparting impact loads to concrete port and harbor structures is the wind. Wind acting upon the structure alone is taken care of within the design of the structure however, when a ship, barge or boat is blown into

the structure, the resulting load imparted into the structure may be significant enough to cause damage and subsequent deterioration to the structure.

Not only do ports and harbors tend to be natural catch basins for all kinds of floating debris, frequently floating objects are dropped into ports and harbors and these objects are then driven by the forces of nature (i.e. tides, wind, currents, etc.) into the structures creating potential sources of damage which will lead to deterioration of the structure. The floating objects may damage the structure by impacting against the structure, abrading against it or destroying small structural members or braces.

Dropped objects frequently cause damage to concrete port and harbor structures. Large loads are moved along, stored upon and staged on port and harbor structures. The movement of these loads is normally accomplished by lifting them above the structure. In accordance with the principles which Newton worked out several hundred years ago, things that go up usually come down and sometimes they come down in unexpected places and at speeds greater than you anticipate. This unanticipated attempted return to earth is the definition of a dropped object. Due to the significant size and weight of some of the objects being

handled over port and harbor structures, dropping them imposes stresses upon the structure which may lead to damage and subsequent deterioration of the structure.

3. Freeze/Thaw Cycle

Temperature differentials can play a significant part in the deterioration of concrete port and harbor structures. The phenomenon known as freeze-thaw cycle can have devastating effects upon concrete port and harbor structures which are located in arctic or even temperate regions of the world.

Due to the ever presence of water on most concrete port and harbor structures, the lowering of the ambient temperature to below freezing will result in ice forming in cracks in the concrete. The resulting ice occupies a larger volume than the previously present water and thus a tensile force is exerted by the ice upon the concrete and may cause propagation of the cracks and continually deterioration of the structure. This phenomenum is illustrated in Figure II.4.

If the rate of deterioration due to freeze/thaw cycles is sufficiently rapid, the reinforcing steel may remain although the concrete is completely wasted away. Figure II.5 is an example of

severe deterioration of a reinforced concrete pile subjected to freeze/thaw deterioration.

Resistance to the deteriorating effects of the freeze thaw cycle can be obtained by providing air entrainment in the concrete by the use of chemical admixtures in the concrete mix. Tiny discrete bubbles of air on the order of two hundredth of an inch in diameter and normally occupying about 6% of the concrete volume, are entrapped within the cement paste. The tiny air spaces provide tensile stress relief for the pressure exerted by the additional required volume of the ice [6].

The freeze/thaw sensitivity of hardened cement paste is also substantially reduced as the water/cement ratio is lowered. A water to cement ratio of less than 0.45 can be utilized within the concrete mix and the entrainment of air will not be necessary. Protection from the harmful effects of the freezing water is provided by the reduction or elimination of the capillary voids which are formed within the concrete by water not fully utilized by the hydration of the cement during the curing of the newly placed concrete [6].

Appendix 2 is a report by V,M Malhotra of CANMET of Ottawa, Canada which gives specific information concerning the durability

of concrete exposed to freezing and thawing. The report goes into considerable detail with regards to testing procedures, developments and results of tests comparing non-air-entrained concrete, air-entrained concrete, air-entrained superplasticized concrete, silica fume concrete and high-strength, air-entrained lightweight concrete.

4. Oceanographic Forces

Lastly, but certainly not least, the oceanographical forces of wind, waves, tides and currents many cause damage to and lead to subsequent deterioration of concrete port and harbor structures. Even though most ports and harbors are designed to provide protection from the forces of the sea (i.e. waves, tides and currents) these forces are still present to some extent. Waves, tides and currents alone do not tend to damage the majority of concrete port and harbor structures, however severe conditions may manifest themselves where these forces become significant. Their unrelenting presence provide avenues for other forms of damage and deterioration as discussed previously in Section I of this report.

C. Chemical Deterioration

Chemical deterioration of concrete port and harbor structures may be the result of attack upon the reinforcing steel, the cement paste or the aggregates contained within the concrete.

1. Corrosion of Reinforcing Steel

Of all the forms of deterioration of concrete port and harbor structures, the most serious form of deterioration is not the disintegration due to the attack on the concrete itself, but rather the electrochemical corrosion of the reinforcing steel.

The results of extensive laboratory testing along with substantial practical experience indicates that the corrosion of the embedded steel can be controlled if only existing knowledge and skill are properly utilized. The disregard of this knowledge and skill will result in the premature deterioration of conctrete port and harbor structures. The process by which reinforcing steel is corroded is illustrated within Figure II.6.

The corrosion of the reinforcing steel imbedded within concrete has a fundamental difference from corrosion in air, namely, the size of the corrosion cell. In air the size of the corrosion cell is microscopic whereas within concrete, the corrosion cell is

macroscopic, the poles of which can be up to several feet apart. Figure II.7 is an illustration of a typical corrosion cell developed along the reinforcing steel embedded within the concrete.

Within this macroscopic zone, chloride ions are deposited by successive wetting and evaporation cycles coupled with the high availability of atmospheric oxygen. Corrosion sufficient to cause spalling of the concrete cover can occur in a relatively short time.

The degradation of both the steel and concrete is dependent on the zone of exposure. The most susceptible region is the splash zone, with the permanently submerged part of the structure being least damaged.

The corrosion of the reinforcing steel has several detrimental effects upon concrete port and harbor stuctures. Corrosion can crack the concrete, with possible spalling resulting in reduced cross section of the member. It can also reduce the effective cross sectional area of the reinforcing steel, and lastly, corrosion can destroy the bond between the reinforcing steel and the concrete resulting in the loss of the structural capacity of the member.

Corrosion of steel can occur via several mechanisms such as direct oxidation, electrolysis, hydrogen embrittlement and electrochemical processes. Corrosion of reinforcing steel within concrete is normally driven by the electrochemical process and it is this form of corrosion which will be discussed herein.

A form of electrochemical corrosion which can occur within concrete results from the significant electrical potential differences which can exist between the different metals or non-uniformities in the steel itself or its surrounding environment. The electrochemical corrosion resulting from dissimilarities within the embedded steel is impeded by a "passivating" iron oxide film which forms on the steel surface during the hydration of the cement. The high alkalinity of the surrounding concrete works in conjunction with the oxide film to provide protection of the embedded reinforcing steel from the electrochemical corrosion process.

Concrete normally provides a high degree of protection against corrosion for the embedded reinforcing steel. This protection is primarily due to the inherent highly alkaline environment which passivates the steel, thus protecting it against corrosion. Also, high quality concrete has a low water to cement ratio and low

permeability which minimizes the amount of other corrosion inducing elements such as oxygen, chloride ions, carbon dioxide and moisture available at the reinforcing steel. Lastly, the low permeability increases the electrical resistivity of the concrete to some degree which assists in reducing the rate of corrosion by retarding the flow of electrical currents that accompany the electrochemical corrosion process within the concrete.

Figure II.8 contains an illustration depicting the amount of time required for initiation of the corrosion process for concrete produced with three different types of cement.

The benign alkaline environment surrounding the embedded reinforcing steel can be attacked and destroyed, however, by two general mechanisms. The first mechanism is the leaching of alkaline substances with water or partial neutralization by reaction with carbon dioxide or other acidic product (carbonation). The second mechanism is an electrochemical reaction involving chloride ions in the presence of oxygen. The importance of the presence of oxygen to the corrosion rate is graphically represented by figure II.9.

The requirement for abundant quantities of oxygen has been verified by numerous researchers who have discovered that

cases of severe corrosion have occurred almost exclusively in the splash zone, and in particular in rectangular deck beams, piles and cross-bracing members [6].

It becomes readily obvious that an effective way of limiting the attack of the reinforcing steel by corrosion can be accomplished by making the concrete as impermeable as reasonably possible. Impermeable concrete can be realized by keeping the water to cement ratio as low 0.40 to 0.45 and insuring adequate fines are available within the concrete mix. Figure II.10 illustrates how the corrosion mechanisms can occur if the concrete is not impermeable.

The corrosion of steel in concrete port and harbor structures is dependent upon the rate of chloride penetration to activate the steel, the resistivity of the concrete and the oxygen diffusion through the cover regions. Reinforcement corrosion may result in the cracking and spalling of the concrete depending on the depth of cover, the physical shape of the member and the strength of the concrete. Figure II.11 illustrates graphically the relationship which exists between chloride concentration and corrosion rate.

Evidence of corrosion of the reinforcing of concrete port and harbor structures can be found in almost all applications. In

most cases, the first indication of corrosion of the reinforcing steel is the presence of brown stains appearing on the surface of the concrete. This brown stain is the result of the corrosion of the reinforcing steel embedded in the concrete and may permeate the surface of the concrete without cracking the concrete however, the brown staining is normally accompanied by cracking of the concrete.

Concrete cracking occurs because the corrosion product of the reinforcing steel occupies a volume twice the size of the original steel. This dramatic increase in volume creates tensile stresses within the concrete which far surpasses the tensile strength of even high quality concrete.

It is believed that complete saturation of concrete in sea water lends protection to the reinforcing steel from the effects of corrosion due to two reasons; (1) the concentration of available chloride ions is relatively low (0.1% by weight of cement) and uniform thereby discouraging the development of differential concentration cells, and (2) the availability of oxygen is also limited as well as impeded in its migration to the imbedded steel by having to diffuse through the saturated concrete cover.

The corrosion of prestressed concrete used in port and harbor structures must be given some additional considerations due to the prestressing to which the imbedded steel is exposed to. If corrosion occurs, the same loss of metal will cause a proportionally greater increase in stress in the prestressing wires. Also, steel under stress has a higher corrosion rate but this effect can be negated by utilizing efficient tendon grouting techniques. Finally, there is a possibility of stress corrosion cracking of the steel, leading to sudden and perhaps catastrophic failure.

Investigations into the few instances of failures of prestressing systems due to corrosion has shown that the principle cause of failure was in maltreatment of the steel before construction and inadequate protection leading to pitting corrosion which formed at sites of subsequent failure [6].

2. Alkaline Attack

Another chemical mechanism which has a deleterious effect upon concrete port and harbor structures is Alkali-Silica reaction. This reaction involves the formation of an alkali silicate gel from the combination of the hydroxyl ions and the sodium or potassium ions present within the cement paste. This gel attracts water molecules which cause a considerable increase in

the internal hydraulic pressure of the concrete. This increased pressure frequently exceeds the tensile strength of the concrete resulting in deterioration of the concrete.

Sodium chloride present in sea water provides excess sodium ions which are the active agents in the alkali-aggregate reaction. The availability of the excess sodium ions can initiate or exacerbate the alkali-silica reaction which produces the expansion and subsequent deterioration of the concrete.

Alkali-silca reaction deterioration may also be the result of evaporation of salt water from horizontal concrete surfaces. In this situatuion, concentric ridge and trough patterns are produced and are suggestive of the Leisegang rings which has been suggested as being involved in the alkali-silica reaction in some concretes containing glassy aggregates [18].

The deleterious effects of the alkali-silica reaction can be avoided by the selection of non reactive aggregate within the concrete mix. Table II.1 is a listing of rock types which are not alkali-silica reactive [18]. This table indicates that only a few rock types with certainty can be considered non-reactive. Recent experience has shown that where igneous, crystalline ore volcanic rocks have been exposed to severe metamorphic

alterations or to weathering induced by long-term tropical heat, they may secondarily have acquired susceptibility to the alkalis in concrete.

For concrete port and harbor structures located in tropical marine environments, the temperature, humidity and salinity are continuous, activating factors which accelerate and aggravate the alkali-silica reaction. In temperate regions the rate of the reactions may be only one fourth of that in the tropics. In concrete port and harbor structures located in arctic regions, alkali-silca reactions may be expected to occur at negligible rates [18].

3. Sulfate Attack

The hardened cement paste binder component of concrete is also subject to chemical attack when used in the construction of concrete port and harbor structures. The chemical attack can be associated with sulphate ions present in sea water reacting with the hydrate of the tricalcuim aluminate present in Portland cements. Obviously, cements which contain low tricalcium aluminate content will be best suited for the concrete of port and harbor structures. The resistance to concrete made with low tricalcium aluminate cement has been tested in the marine environment and has proven to more durable [6].

The American Society for Testing Materials (ASTM) standard specification for Portland cement (# 150-71 (1971)) classifies ordinary Portland cement as type I and sulphate resisting cement as type V. Type V Portland cement has a maximum permitted tricalcium aluminate content of 4%, however type II cement, which has a maximum permitted tricalcium aluminate content of 8%, is recommended for concrete exposed to the marine environment by the ASTM standard. Type II Portland cement is considered to have moderate durability but has proven to be highly acceptable in both laboratory test and by surveys of existing concrete structures exposed to the marine environment. The effect of tricalcuim aluminate content is illustrated in Figure II.12 [6].

Further protection against sulfate attack can be afforded by the inclusion of slag or pozzolanic materials as a partial replacement for the cement in the concrete mix. Several types of slag are suitable for concrete exposed to the marine environment but the most popular type is ground blast furnace slag. This slag can be used to replace typically between 20% and 60% of the cement, by weight, in the concrete mix.

The slag used is of a siliceous nature, and hydrates to form a cementitious material compatible with the Portland cement and can

be considered as becoming part of the cement content, effectively reducing the amount of tricalcium aluminate content of the concrete mix. Additionally, the inclusion of the blast furnace slag will lower the heat of hydration of the concrete which may be desirable if massive amounts of concrete are being placed [6].

As can be clearly seen, all the chemical attacks which the concrete of port and harbor structures must endure is related to the absorption, in one way or the other, of water or ions into the concrete. Therefore, the importance of low permeability, porosity and absorption cannot be over emphasied with respect to concrete port and harbor structures.

D. Biological Deterioration

Concrete port and harbor structures may suffer deterioration due to biological organisms. The deterioration may be a direct attadk upon the concrete structure or may degrade the structure by overloading the structure.

1. Biofouling

Deterioration to concrete port and harbor structures by biological organisms is limited primarily to the boring clam Lithophaga,the sea urchin and the rock-boring mollusk, Pholadidae [38]. The attachment of the shells of the organisms exert high pressures on the surface of the concrete structure as the organism grows. This pressure increases as the organism matures from the embryonic stage to adulthood. This pressure may damage the concrete and destroy any protective coating or membranes which may have been applied to the structure [58].

The rock boring mollusk is only able to bore into poor, weak concrete. The boring clams secrete an acidic material which can dissolve the cement and are most aggressive and destructive on porous concrete [38].

Marine organisms which pose a threat to the performance of concrete port and harbor structures are most after found in tropical and subtropical environments. Very aggressive mollusks exist in the Arabian-Persian Gulf which can bore into the hard limestone of high-strength concrete [58]. Their attacks are normally effective only on weak, porous and soft concrete. A hard, dense concrete surface will usually provide adequate protection against the deteriorating effects from marine organisms [38].

Marine growth is influenced by temperature, oxygen content, pH, salinity, current, turbidity, and light. While the majority of growth takes place in the upper 60 feet of the water column, significant growth has occasionally been found at three times this depth [58].

Anaerobic sulfur-based bacteria are often trapped in the sediments which may be adjacent to concrete port and harbor structures. When the sediments are disturbed, these bacteria can be released into the sea water and are converted to sulfates. Upon subsequent exposure to the atmosphere, they become sulfides. Theobacillus concretivorous bacteria attack weak and permeable concrete resulting in damage and subsequent degradation [58].

2. Overloading

Along with the actual destruction of the concrete of port and harbor structures, marine organisms can contribute to additional deterioration by increasing the dead load on the structure as well as increasing the drag force which may be exerted on the structure. Barnacles and mussels increase the effective diameter of concrete piles and more importantly, the surface roughness. Because of this latter, the drag coefficient used in the Morrison's equation, can be increased by 100 percent. Fortunately, the additional mass applied to a concrete port or harbor structure by marine organisms is negligible due to the fact that most marine organisms have a specific gravity very near that of the surrounding seawater [58].

E. Design Deficiencies

As if the environment in which concrete port and harbor structures are placed and expected to function is not a great enough challenge, deterioration occurs due to deficiencies in the design of the structures.

1. Concrete Cover

Sufficient cover of reinforcing steel must be provided for in order to protect the structure from the deterioration effects of corrosion. Figure II.13 illustrates the peneratration rates of sea water into concrete and indicates the concrete cover required [6].

Special attention must be paid to detailing the form design and construction to ensure that a minimum of 2 inches of concrete cover is provided, at ALL locations. Tolerances associated with the placement of the reinforcing must be taken into consideration and the required cover provided in order to overcome these circumstances.

Upon removal of the formwork, any "honeycombing" which exposes the reinforcing steel should be patched immediately in order to avoid the creation of corrosion cells.

2. Shapes of Members

As importantly as providing sufficient cover to the reinforcing, concrete port and harbor structures should be designed with members which do not promote deterioration by their shape or orientation. Browne [6] gives several examples of structural shapes which will combat the deterioration effects of the environment, Figure II.14.

Special attention should be paid to ensure that pockets and ledges where water and debris can accumulate are avoided. If pockets and ledges do exist, the accumulation of seawater with subsequent evaporation will provide excess chloride ions which will lead to the corrosion of the reinforcing steel and deterioration of the structure. Figure II.15 illustrates the relationship between shape of the concrete member and the relative degree of degradation which can be expected over time.

F. Construction Deficiencies

1. Quality Control/Quality Assurance (QC/QA)

Along with design deficiencies, construction deficiencies also contribute to the premature deterioration of concrete port and harbor structures. As stated throughout this section, most of the deteriorating effects upon concrete port and harbor structures can be negated by a conscience and continual effort of insuring quality of construction. Insurance of quality of construction can only be assured by the efforts of competent and fully supported QC and QA. Careful and considerate design can be completely negated if proper QC/QA is not an ever present function at the work face. The requirements upon the QC/QA staff are monumental yet entirely necessary. The QC/QA staff must have full authority from project management in order to ensure that quality is constructed into the structure. Without this authority and support from the highest levels of management, all the best efforts at producing quality concrete port and harbor structures which will endure the rigors of the marine environment will be for naught.

2. Salt in Mixwater

One of the most common construction deficiencies which can occur during the construction of concrete port or harbor

structures is the intentional or inadvertent addition of saltwater into the concrete mix. This practice is paramount to asking the fox to guard the henhouse! The advent of corrosion is enhanced and it will only be a matter of time, and a short amount of time at that, before the deteriorating effects of corrosion will become noticeable.

3. Curing Time

Due to the loads placed upon concrete port and harbor structures, it is imperative that all newly placed concrete has sufficient cure time. Premature exposure and loading of green concrete is likely to set up deterioration mechanisms (i.e. micro and macro cracking) which may be manifested throughout the structure. Some of these mechanisms may not be readily noticeable and the result will be a concrete port or harbor structure that does not provide the level of service to which it was designed.

G. Conclusion

The magnitude of deteriorating mechanisms which concrete port and harbor structures are exposed to has been stated within this section. It is imperative that specific precautions be taken during the design, construction and maintenance of these structures in order to ensure their satisfactory performance. Quality concrete port and harbor structures can be obtained by adhering to the following list of recommendations ;

1. Material Selection:

   A. Cement - Portland Cement, ASTM Type II, moderate tricalcium aluminate (5-8%), low alkali content, uniform quality.

   B. Aggregates - ASTM C33, uniform grading, fully tested, satisfactory past history of sea water durability and freeze-thaw durability (if applicable), resistant to sulfate attack, free from chlorides (<0.02% Cl by weight), nonalkali reactive, control content of fines to ensure stability at high slump.

   C. Water - Cl < 500 ppm, Sulfate Content < 1000 ppm.

D. Admixtures - Ensure compatibility of all admixtures with the other constitutes of the mix, use no admixtures containing chlorides, select efficient water reducers, air entrainment (6%) for freeze-thaw durability (if applicable).

2. Mix Design :

A. Water/cement ratio <0.45 (<0.40 in splash zone locations).

B. Cement content of 7 sacks/cu yd (preferably 8 sacks if possible).

C. Full scale site trials before mix is chosen.

3. Mixing Procedures :

A. Employ modern automatic batching plants for effective and efficient mixing.

B. 100% QA on batch records.

C. Maintain constant check on workability.

D. Ensure back-up plant and reserve materials

4. Placement :

A. Prepare and enforce strict placement procedures.

B. Thoroughly consolidate by high frequency vibration to eliminate all "honeycombs", rock pockets and "bug holes".

C. Revibrate top layers during slip forming operations.

D. Ensure cover of at least 2 inches and fully compacted.

E. Deficient cover is compensated by a cement based grout coating or epoxy coating.

F. Surface finish should be smooth, dense and free from imperfections.

G. Construction joints - clean off laitance with water jet or sandblast to expose coarse aggregate, prime with epoxy bonding agent just prior to next lift and apply rich mix against joint.

5. Curing :

A. Water cure with fresh water for seven days followed by 14 days of drying prior to immersion in sea water.

B. If steam curing is utilized, follow with one day of fresh water curing.

C. Avoid curing temperatures above 160 degrees F and reduce temperature gradients across cross section.

D. If young concrete must be exposed to sea water, it must be positively protected by a water-tight membrane or sheath.

6. Grouting :

A. Use a neat cement-grout with a water/cement ratio of 0.45 maximum.

7. Design :

A. Perform constructability review from a concrete placement viewpoint (keep design as simple as possible).

B. Provide for larger sections to aid in placements.

C. Rounded and smooth surfaces should be utilized as much as possible.

D. Avoid sudden changes in geometry.

E. Larger rebars provide for easier placement and the use of T-headed bars reduce stirrup congestion.

F. Cathodic protection of adjacent steel structures should be limited to the use of sacrificial anodes.

8. Personnel :

A. Use trained and qualified operators and constructors.

B. Provide for continuous and effective quality control and assurance.

Appendix 3 has been included to provide various recommendations to follow in order to produce concrete structures which will function in the marine environment without undue maintenance and/or repair due to deterioration of the concrete.

TABLE II.1 Rock types which are not Alkali-Silica reactive

LIMESTONE	Calcium carbonate without magnesium and silca
BASALTS	Crystalline, low silica
GNEISS, GRANITE	Without crushed and strained quartz or secondary weathering
SAND	Of above rocks, also pure quartz

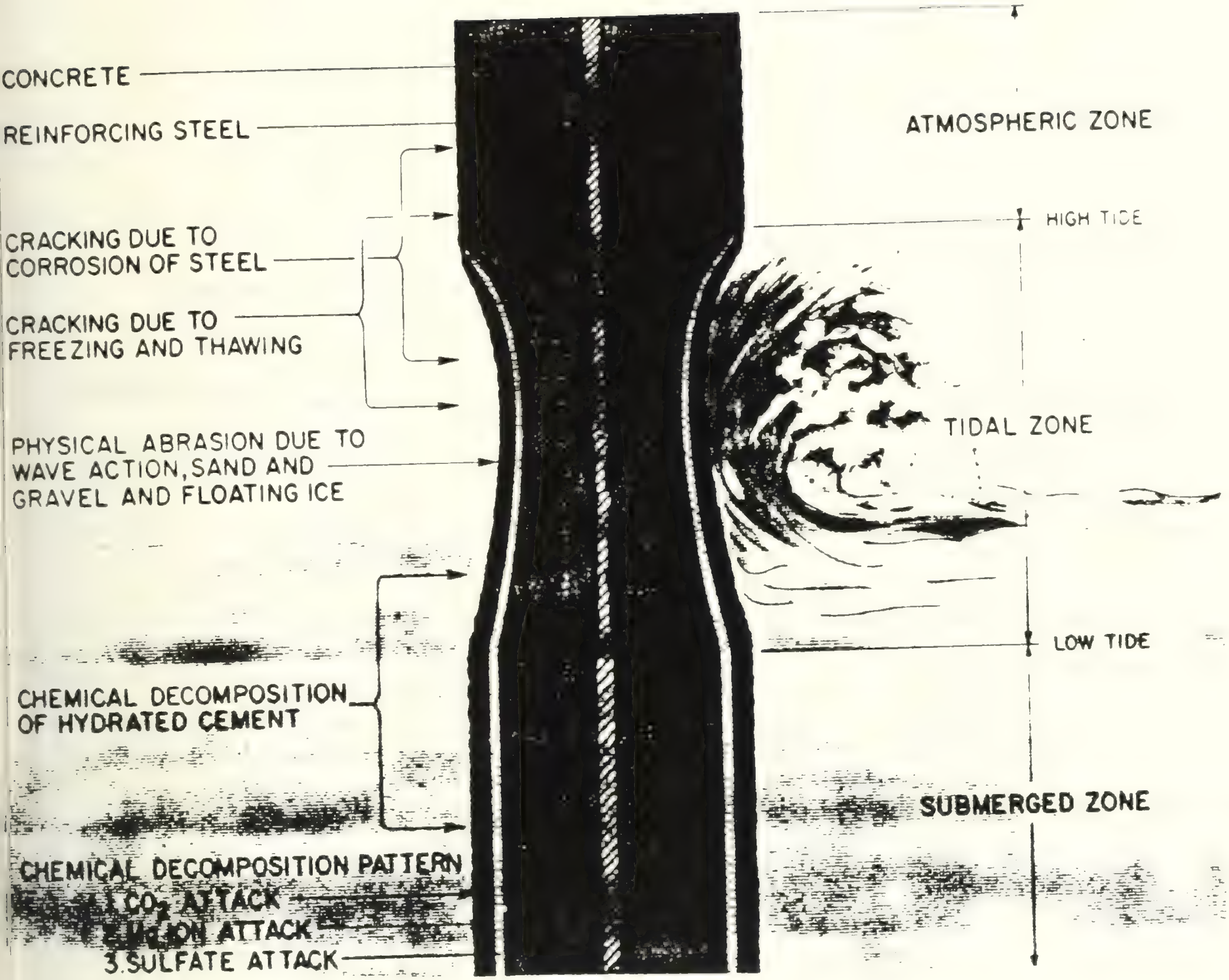
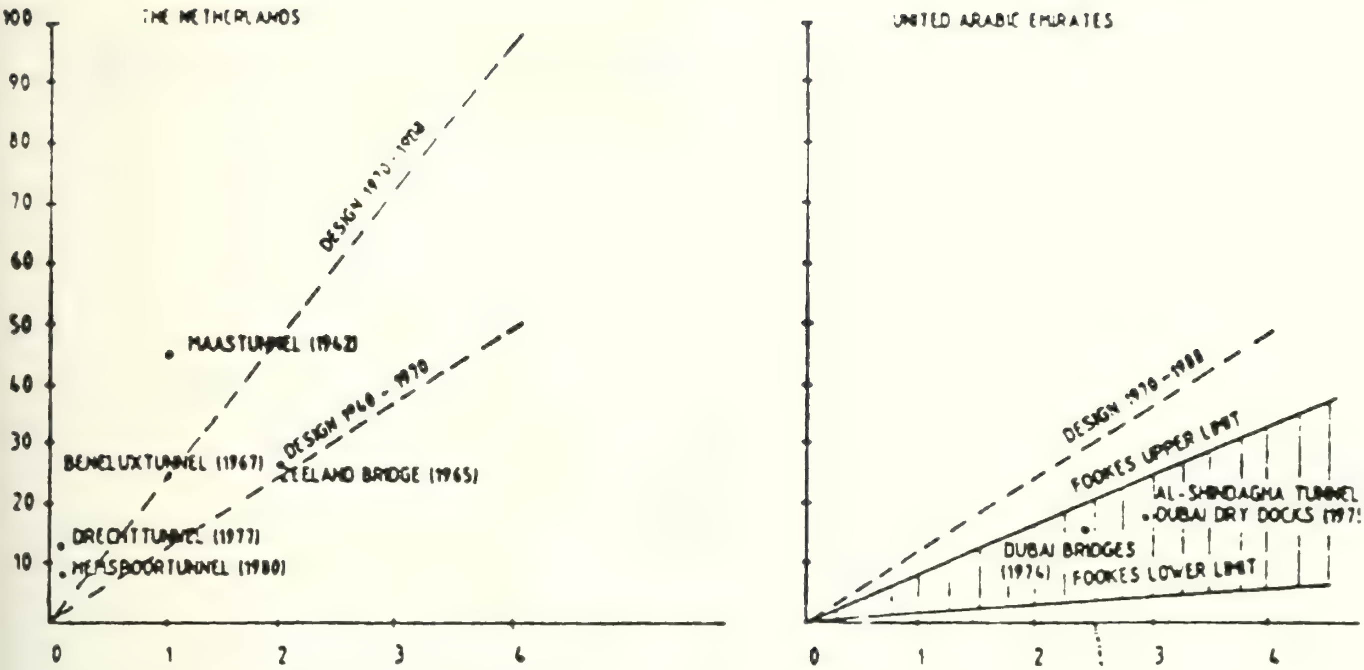


FIGURE II.1 Processes responsible for deterioration of reinforced concrete exposed to the marine environment



CLASS	0	1	2	3	4
REINFORCED	NO DEFECTS	RUST STAINING AND MINOR CRACKING	ONSET OF CORROSION CRACKING	MODERATELY SEVERE CRACKING SOME SPALLING	SEVERE CRACKING AND SPALLING TERMINAL CONDITION
MASS	NO DEFECTS	MINOR SURFACE WEATHERING OR POP - OUTS OR CRACKS	MODERATE SURFACE WEATHERING ISOLATED CRACK	MODERATE TO SEVERE WEATHERING OR INTERCONNECTED CRACKS	SEVERE LOSS OF CONCRETE SURFACE DISRUPTIVE CRACKING TERMINAL CONDITION

FIGURE II.2 Effects of heat as a deteriorating factor for concrete exposed to the marine environment

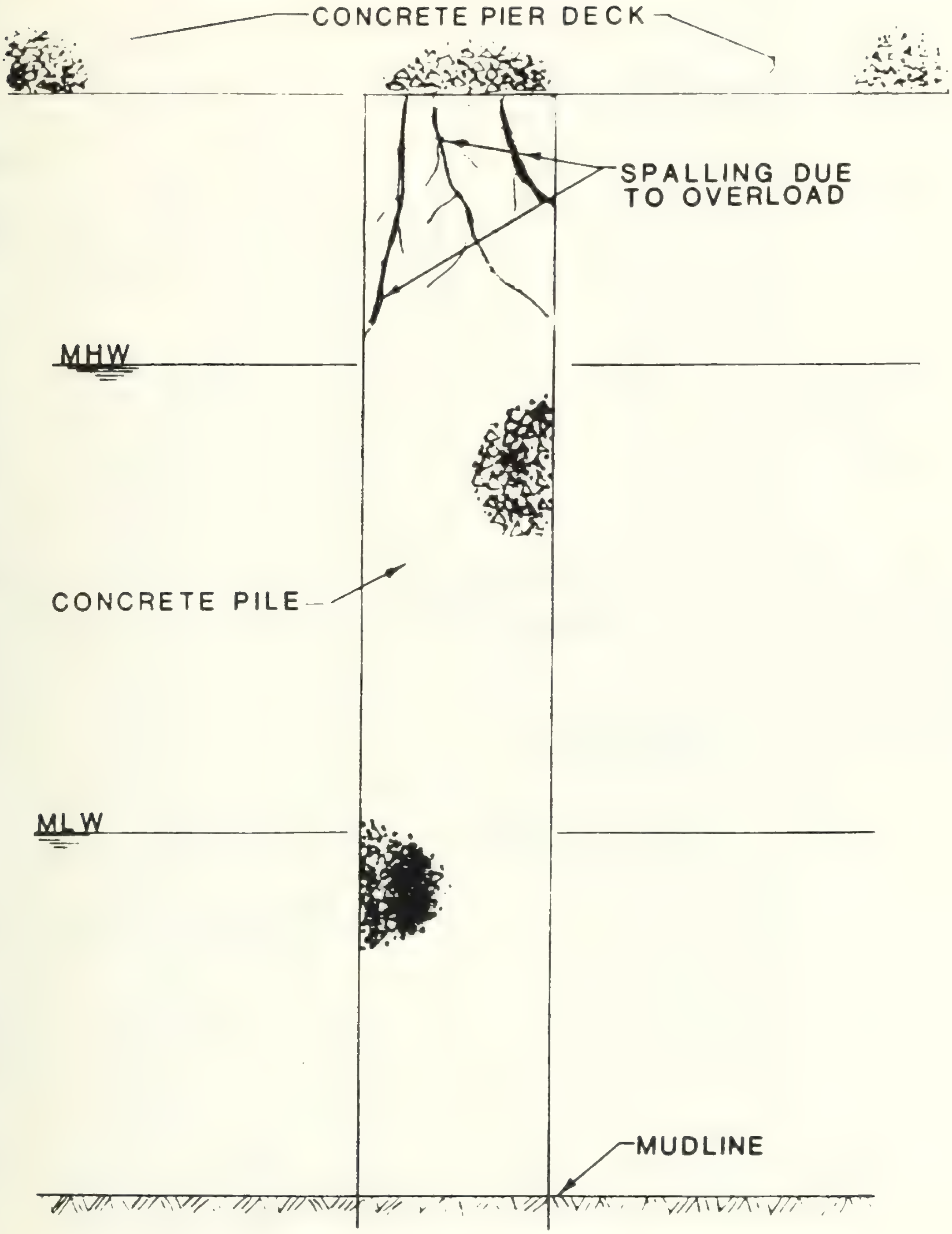


FIGURE II.3 Deterioration of a reinforced concrete pile subjected to overloading

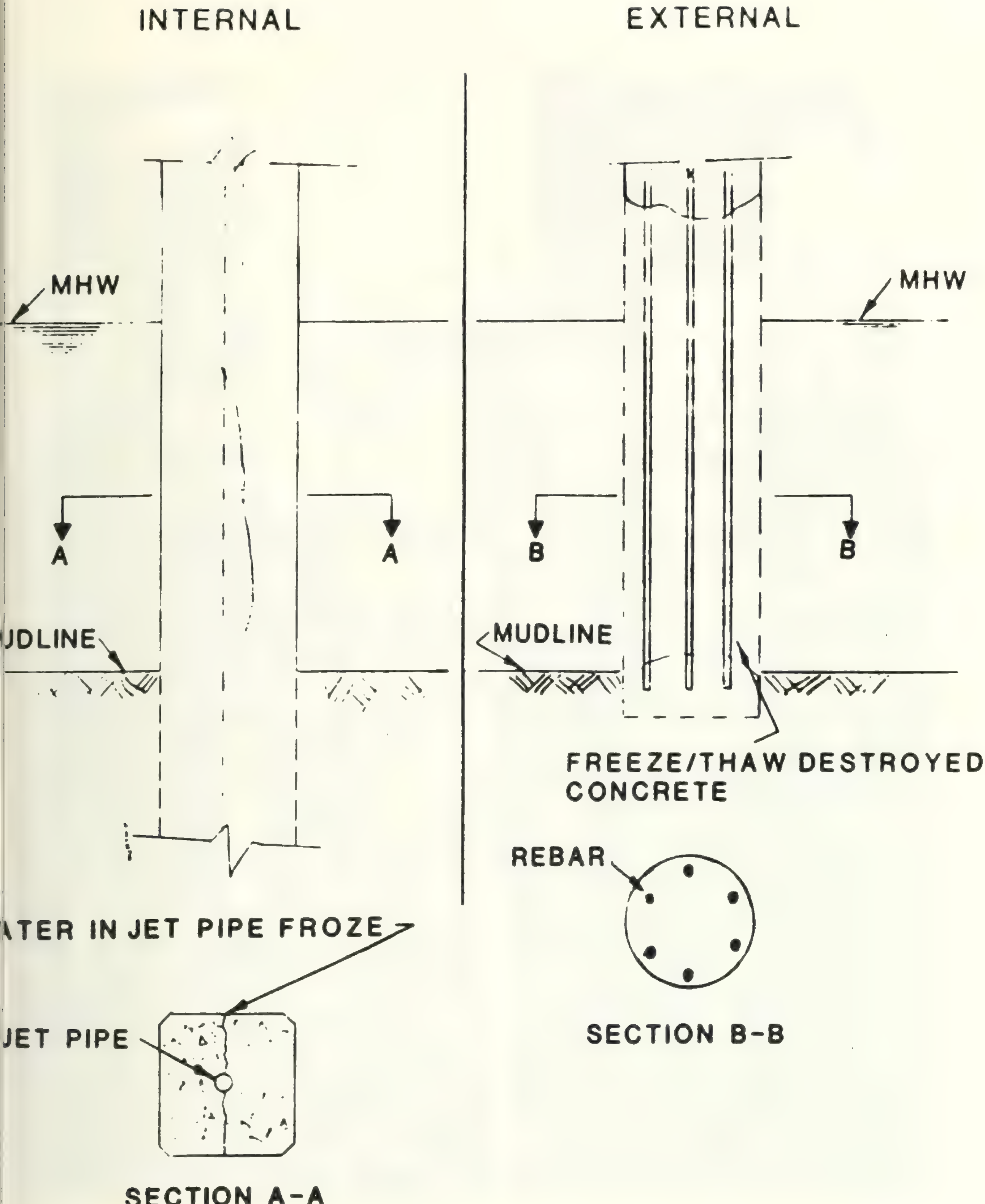


FIGURE II.4 Deterioration of reinforced concrete piles due to freeze/thaw cycle

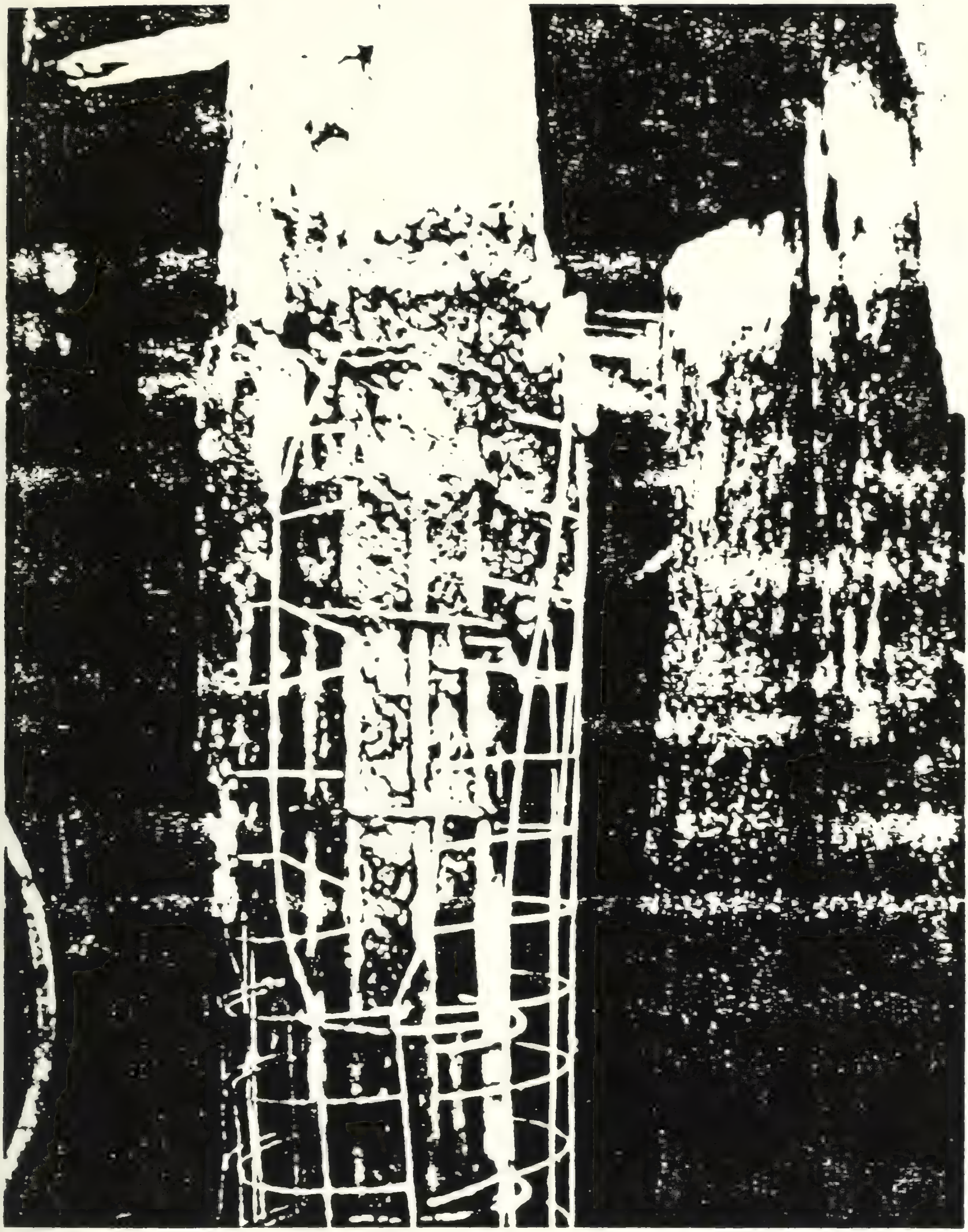


FIGURE II.5 Deteriorated reinforced concrete pile due to freeze/thaw cycle

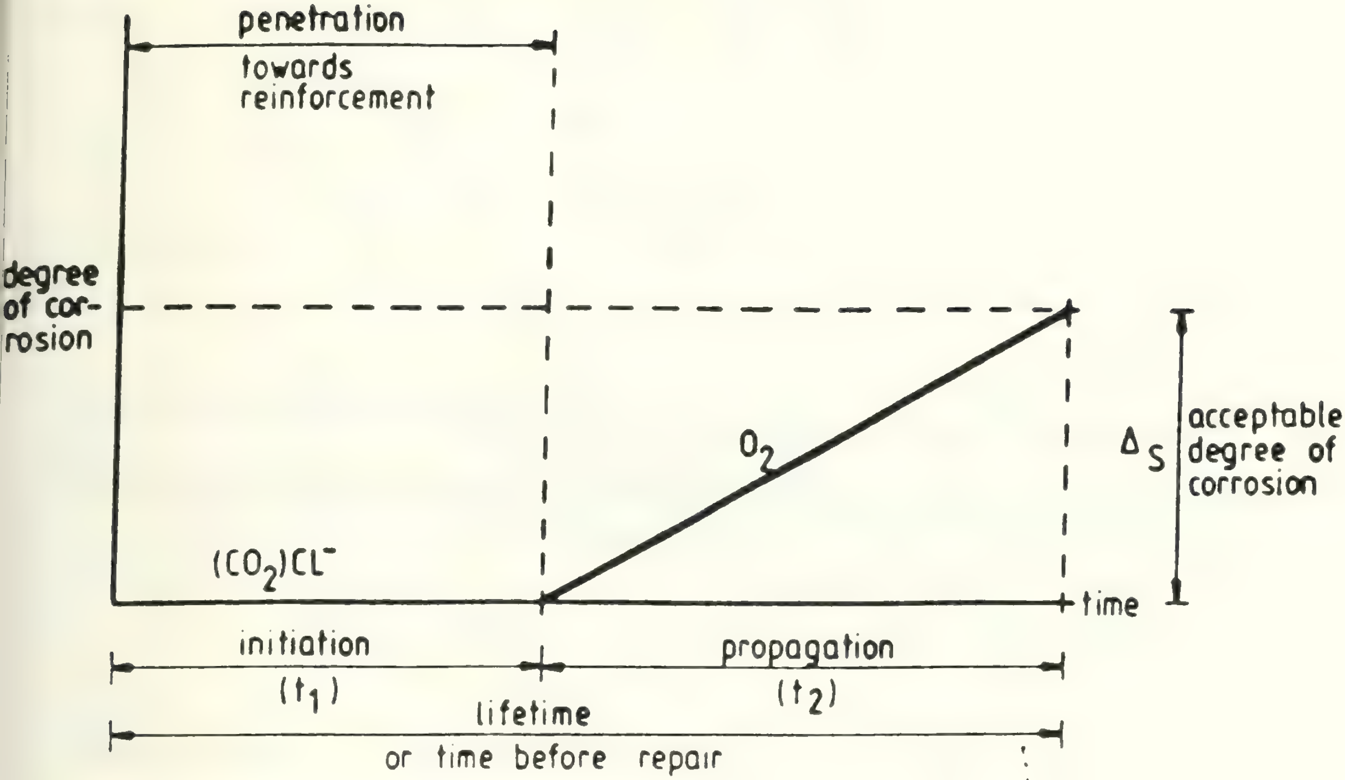


FIGURE II.6 Schematic representation of the corrosion process of steel in reinforced concrete

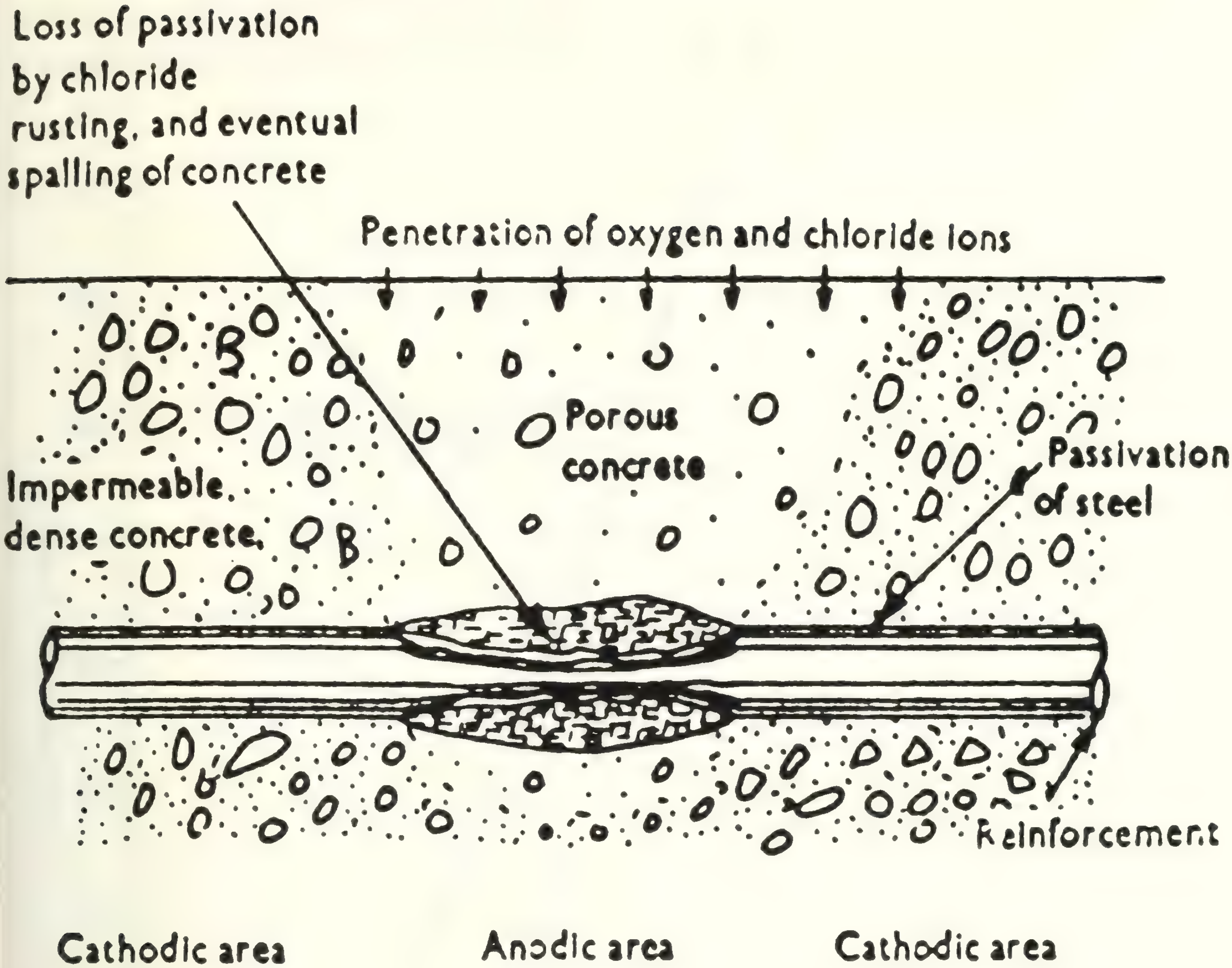


FIGURE II.7 Typical corrosion cell

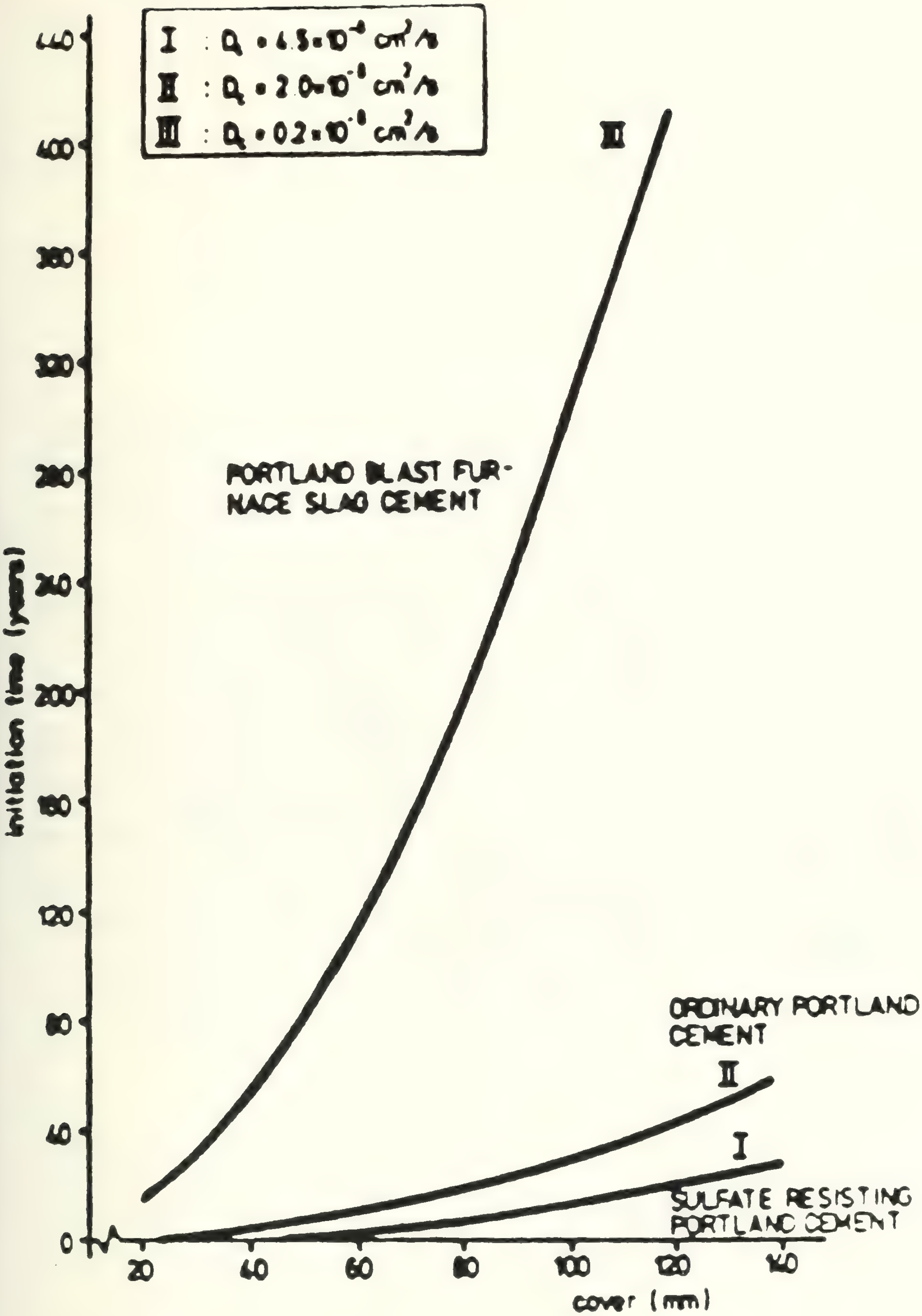


FIGURE II.8 Time for initiation of the corrosion process for three different types of cement

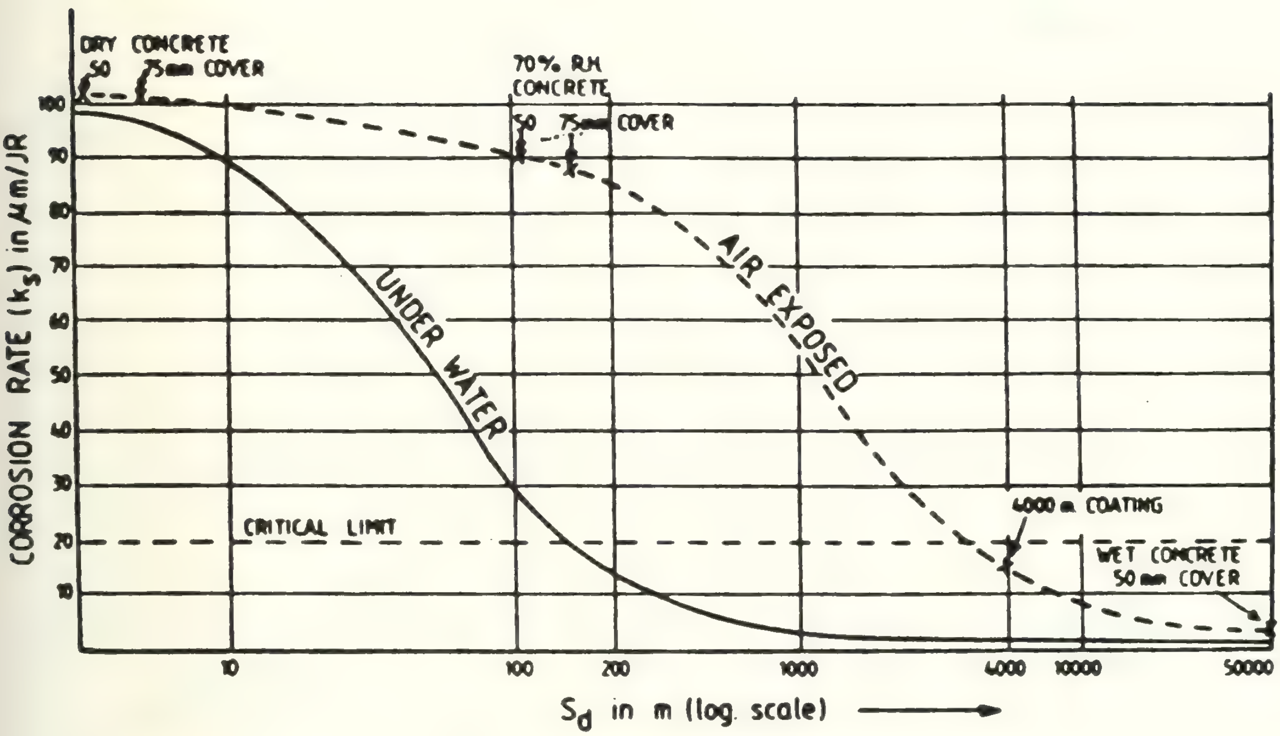


FIGURE II.9 Corrosion rate as a function of the oxygen diffusion resistance

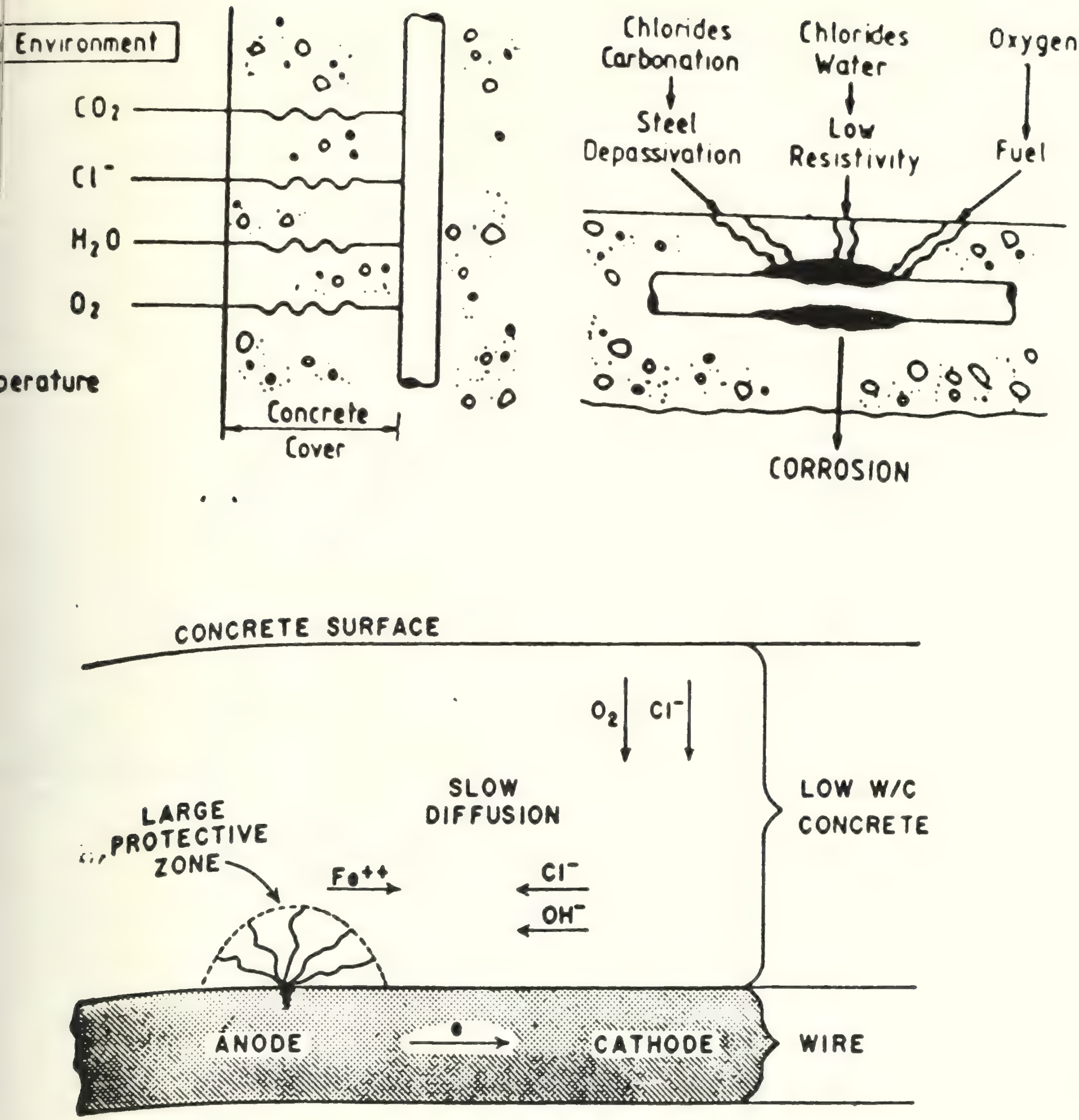


FIGURE II.10 Mechanisms of corrosion

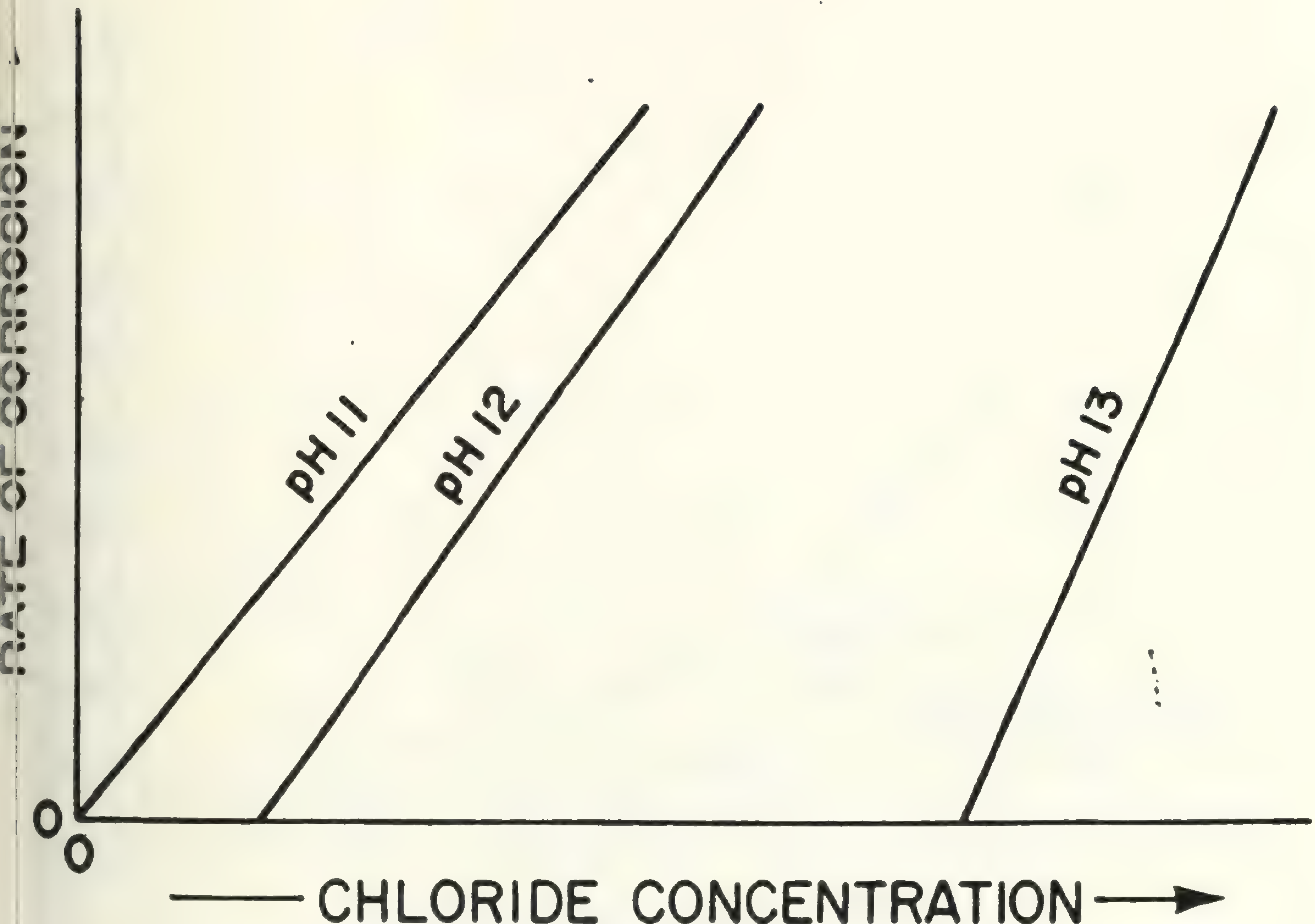


FIGURE II.11 Relationship between chloride concentration and rate of corrosion

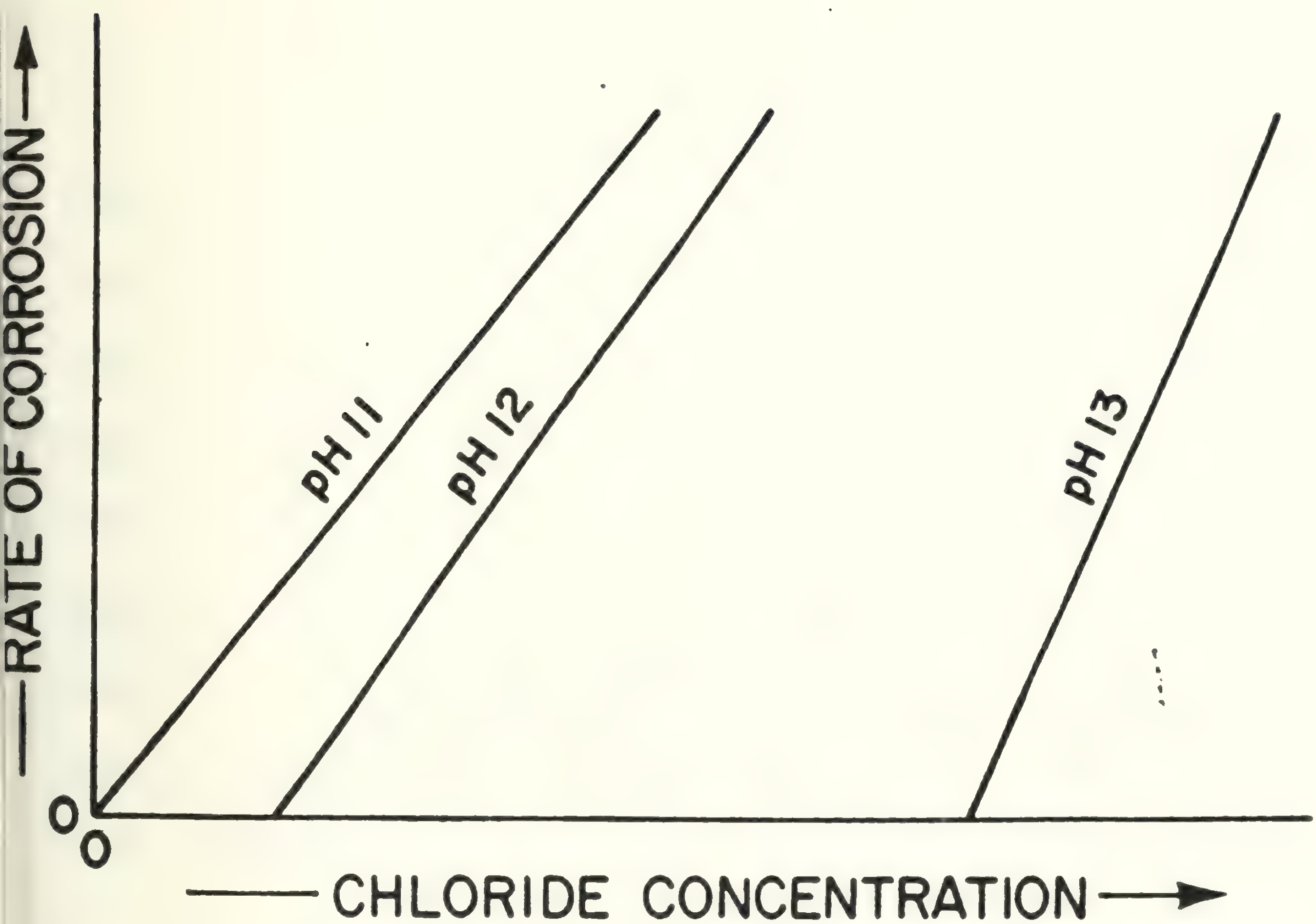


FIGURE II.11 Relationship between chloride concentration and rate of corrosion

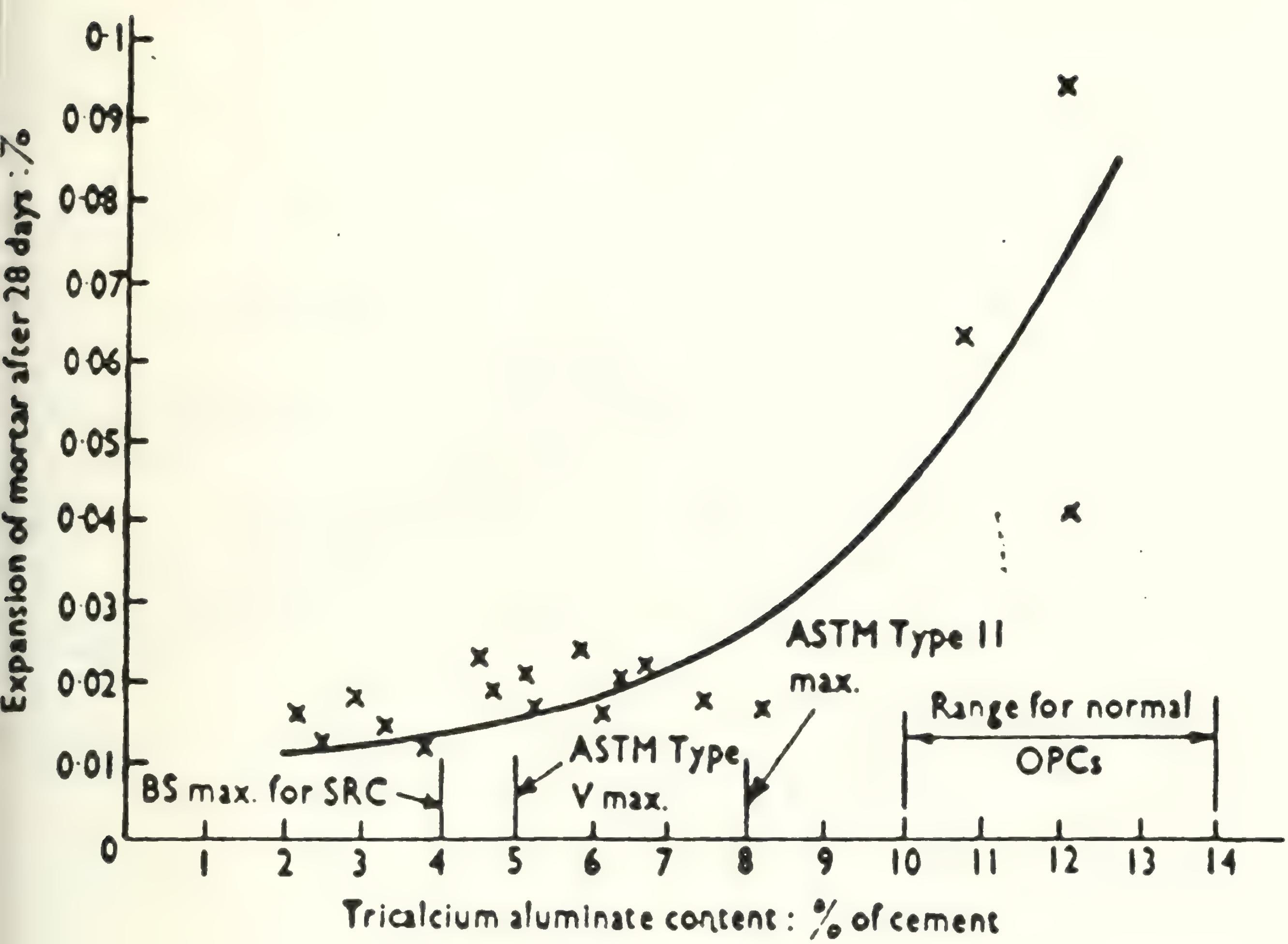


FIGURE II.12 Effect of tricalcium aluminate on sulfate resistance of Portland cements

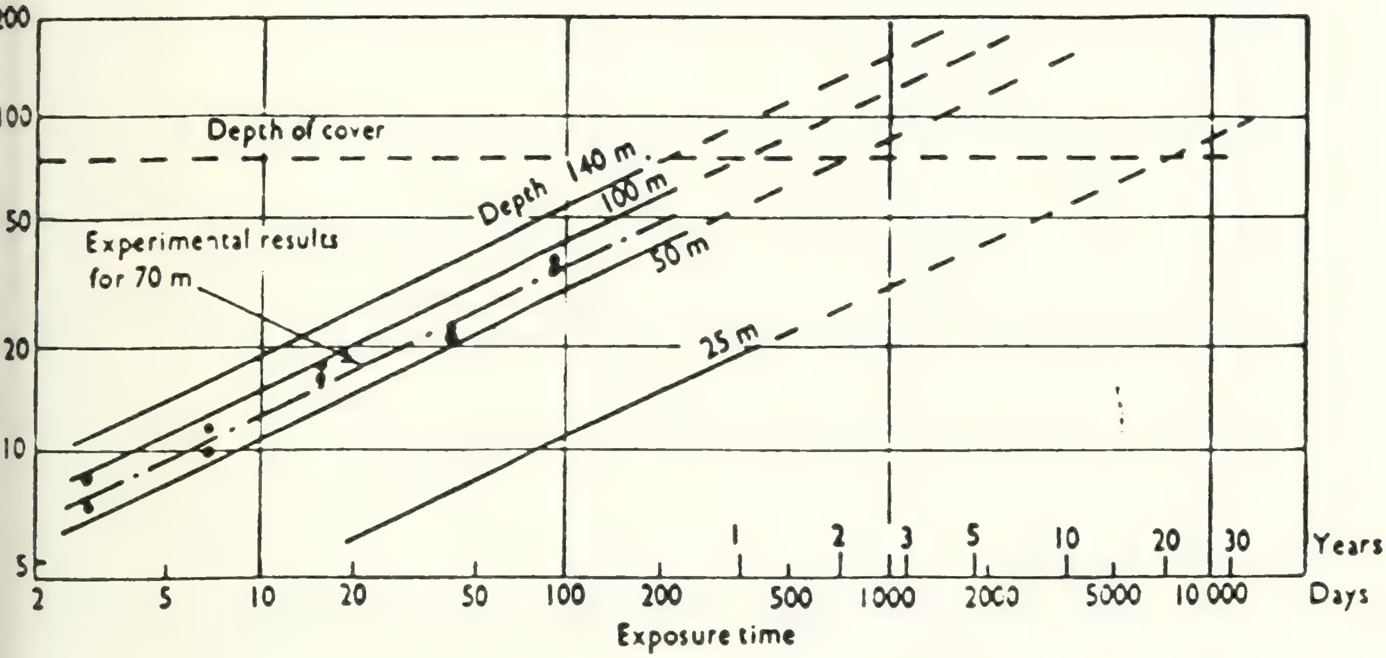


FIGURE II.13 Penetration rates of seawater into concrete at various depths

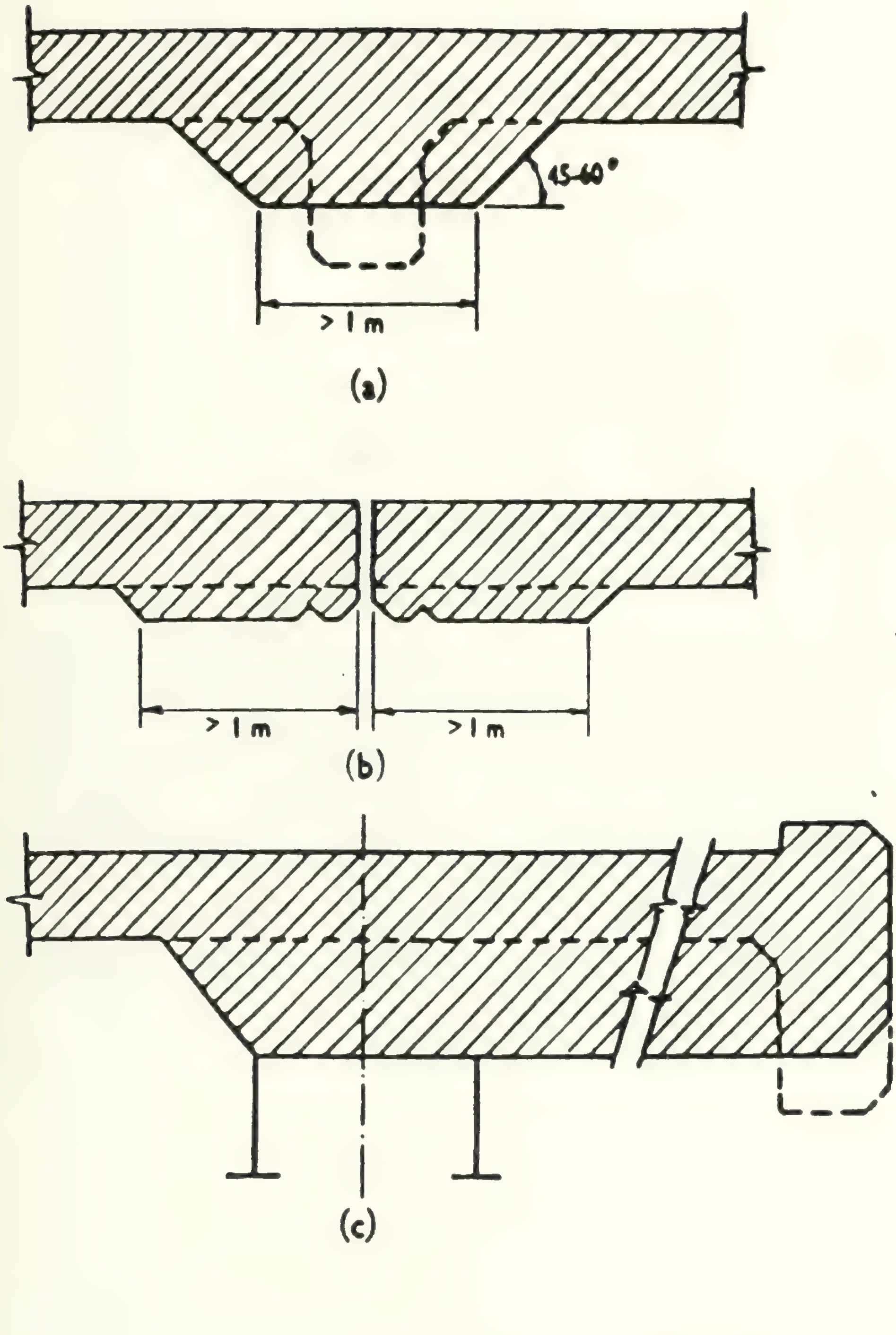


FIGURE II.14 Recommended shapes for structural members

Traditional designs are represented by dotted lines

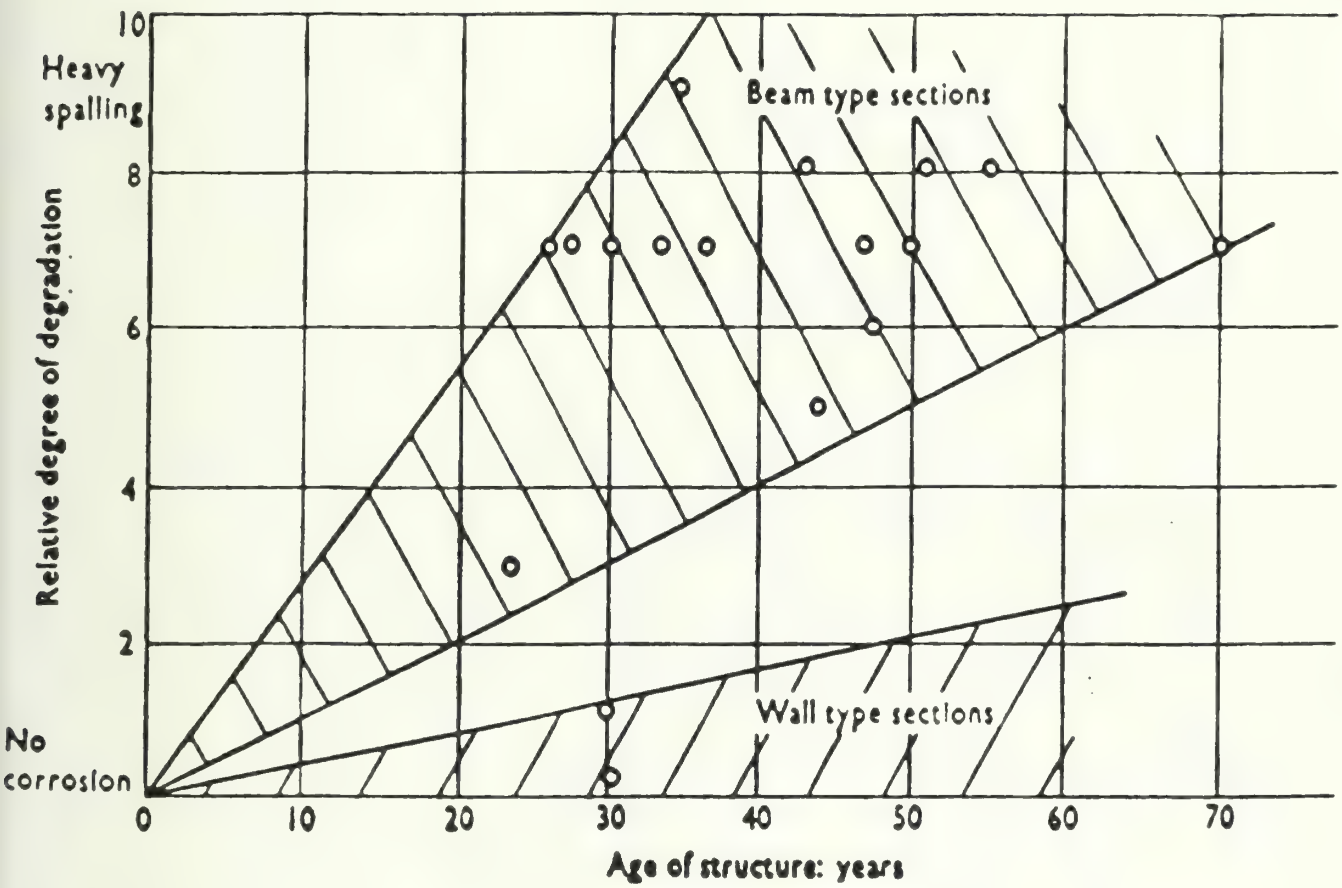


FIGURE II.15 Shape of structural member effects upon the degree of degradation verses age of structure

SECTION III

ASSESSMENT OF DETERIORATION OF CONCRETE PORT AND HARBOR STRUCTURES

by

Jim Schofield

A. Inspections

1. General

It is necessary to make regular periodic inspections of a concrete port and harbor structure in order to assess the ability of the structure to carry out the function for which it was designed, and to assess the need for ongoing maintenance and repair. The time between inspections is dependent upon the service use of the structure, and varies considerably depending on cost, and the criticality of the operation of the facility. A typical period between inspections may be on the order of five years. The U.S. Navy goal is a six year inspection cycle, and as with most facility owners, that cycle is heavily dependent on funding, repair priorities, and manpower.

In any effort to assess the condition of the structure as listed above, the inspection of the structure should be designed to provide the following information:

1) the present state of the structure, including information on deterioration and the effect of the environment.

2) the future rate of deterioration and environmental effect and likely life

3) the maintenance/repair necessary to maintain the design or amended design life. [42]

Of special interest in item (3) above is the phrase "amended design life". In plainer terms, this is often a diplomatic way of saying that no money exists for new construction so repairs better hold things together.

At the outset of an inspection effort, the task starts with a historical perspective. It is important to obtain as much historic information as possible to educate the inspectors and augment their ultimate findings. Such historical information may include drawings, original specifications, environmental conditions, design criteria, construction information, details of previous inspections, and documentation of previous repair or maintenance efforts.

Commencing with the collection of historical data, a typical inspection may include (in ascending order of cost, detail, and complexity) :

--- collecting all available historical data

--- visual inspection

--- non-destructive on site testing

--- laboratory testing

--- destructive testing

In addition to these general levels of inspection, any given concrete harbor facility may be divided for inspection purposes into five inspection zones. First is the atmospheric zone which is that part of the structure that is high enough above the influence of marine environmental conditions to treat it as one would regard a typical 'dry land" structure. The second zone is the most scrutinized portion of the structure, the splash zone that exists between the high and low water marks as defined by the extreme limits in elevation of seawater influences including water entrained in the air and vice versa. This is a broader zone than the third zone which is the tidal zone, which is narrower than the splash zone and described by the range of the tides between mean higher high water and mean lower low water. (See section I of this report for more detailed description of tidal ranges.)

Below the splash zone is the submerged zone which is constantly submerged and thus less subject to atmospheric influences.

Below the splash zone is the mudline interface zone, which is that portion of the structure that extends into and below the mudline. For some structures, such as concrete piles repaired in this case study, the splash zone may encompass most of the structure from the mudline up to the high water mark. In such a case there are no structural members that remain constantly submerged. Figure (III.1) shows schematically the five inspection zones for the case of a concrete pile.

There are several ways to categorize actual levels of inspections. The U.S. Navy utilizes a three level system which is summarized here [61]:

DEFINITIONS : The following are definitions of standard levels of effort required to be exerted in each inspection. The scope of work for all inspections break down the total inspection effort into these levels and specifies the amount of work required in each level. The procedures prescribed for most inspections are commonly a combination of at least two of the levels of examination. The terms Level I and Level II, etc., are referred to frequently in a Scope of Work for an inspection report.

LEVEL I : General Examination This level of effort is essentially a "swim-by" overview, which does not involve cleaning of any structural elements, and can therefore be conducted much more rapidly than the other levels of examination. The Level I examination should be used to confirm as-built structural plans and detect obvious major damage or deterioration due to overstress (ship impact, ice loading, etc..), severe corrosion or extensive biological growth and attack.

The underwater inspector will rely primarily on visual and/or tactile observations (depending on water clarity) to make condition assessments. These observations are normally made over the total exterior surface are of the underwater structure whether it is a quaywall, bulkhead, seawall, pile or mooring.

Visual documentation (utilizing underwater television and/or photography), may be included with the quantity and quality adequate for documentation of the findings which will be representative of the facility condition.

The results of the Level I inspections should be evaluated to determine the required amount of additional inspection which may be required. Level I inspections are recommended after the occurrence of any event which may be expected to cause damage

or even undue stress on the facility. The use of remotely operated vehicles is becoming more popular in the conduct of Level I inspections. Figures (III.2) and (III.3) are photographs of divers engaged in a typical Level I inspection. Note the significant buildup of marine growth on the structures.

MODIFIED LEVEL This level of effort consists of a "swim-by" of every pile at an elevation of two to four feet below mean low water line to detect any obvious gross or major damage.

LEVEL II : Detailed ExaminationThis level of effort is directed toward detecting and indentifying damaged/deteriorated areas which may be hidden by biofouling organisms or surface deterioration. At this level, a limited amount of measurements may be made. These data should be sufficient to permit estimates of facility load capacity. Level II examinations will often require cleaning of structural elements. Since cleaning is time consuming, it is generally restricted to areas that are critical or which may be representative of the entire structure itself. The amount and thoroughness of cleaning to be performed is governed by what is necessary to discern the general condition of the facility. Simple instruments such as calipers, measuring scales and ice picks are commonly used to take physical measurements. However, a small percentage of more accurate measurements may

also be taken with more sophisticated instruments for several reasons. These will validate large numbers of simple measurements and in some hard-to-measure areas will actually be easier and faster to obtain. Where visual scrutiny, cleaning, and/or simple measurements reveal extensive deterioration, a small sampling of detailed measurements will enable gross estimates to be made of the structure's integrity. For example, on extensively deteriorated concrete piles with obviously corroded reinforcing steel, a small percentage should receive ultrasonic thickness measurements to determine the typical cross section profiles. The cross sections determined by these spot checks would be used to determine individual load capability which would then be extrapolated to obtain a "ballpark" estimate of overall facility load capability.

Visual documentation (utilizing underwater television and/or photography) should be included with the quantity and quality adequate to be representative of the range of facility damage/deterioration.

<u>LEVEL III : Highly Detailed Examination</u>This level of effort will often require the use of Non-Destructive Testing (NDT) techniques, but may also require the use of partially destructive techniques such as a sample coring through concrete

and wood structures, physical material sampling, or in-situ surface hardness testing. The purpose of this type of evaluation is to detect hidden or interior damage, loss of cross-sectional area and material homogeneity. A level III examination will usually require prior cleaning. The use of NDT techniques are generally limited to key structural areas, such as areas that may be suspect or to structural members which may be representative of the underwater structure.

Visual documentation (utilizing underwater television and/or photographs) and a sampling of physical measurements should be included with quantity and quality adequate for documentation of the findings which will be representative of the facility condition. Figure (III.4) shows a representative pile inspection form which provides spaces for recording all of the inspection data in tabular form. Figure (III.5) provides abbreviated condition ratings for the inspector's use on concrete piles. Similar ratings may be used for other structural elements.

Any or all of the levels of inspection described above may be necessary on different zones of varying structures. It is the task of the engineer to decide how much of the structure to inspect and to what level of detail. It is typical for repetitive structures for an underwater inspection to cover 10 - 30 % of

the structural members. This approach may be refined, however, with a knowledge of high stress areas or historical information which would lead the engineer to scrutinize certain areas closely while inspecting other areas only superficially.

Figure (III.6) tabulates typical defects that the engineer may consider "detectable" by each of the three levels of inspection.

One method utilized to determine inspection sample size is Bayesian updating. This is a probabilistic technique in which sample sizes are influenced by past inspection results to gain a more representative picture of the overall facility. Appendix 4 is a technical paper which summarizes the use of Bayesian updating for underwater inspection planning.

2. Inspector Training

Inspection reports are only as good as the individuals carrying out the inspection procedures. Qualification as a recreational SCUBA diver does not qualify one to be an inspector of underwater facilities. The diving conditions for underwater inspections (poor visibility, underwater obstacles, and unpredictable currents) are very different from those for sport diving.

Commercial dive training (including U.S. Navy) and experience are very desirable preparation for the conditions of underwater inspection. Minimal requirements for underwater inspectors consist of diving certification through a nationally recognized training agency, physical fitness to dive attested by a physician knowledgeable in underwater medicine, experience diving in limited or no visibility, and recent diving activity. It is essential that the diving activities be overseen by an experienced divemaster. NOAA and Navy regulations offer guidelines for diver qualification. [48]

3. Inspection Methods

There are a myriad of inspection techniques , some of which are mentioned in the summary of inspection levels above. The following sub-sections describe in approximate order of complexity many of the inspection techniques in use today for the inspection of concrete port and harbor structures. Figure (III.7) is a tabulated summary of current underwater NDT techniques , including their advantages and limitations. Note that in the table, "visual" methods encompass many of the in-situ, relatively simple techniques including those that require some simple hand tools.

a. Visual

The unaided human eye remains the most efficient and effective inspection tool in our inspection inventory. It is capable of identifying blemishes, cracks, stains due to corrosion of reinforcing steel, and other self evident forms of deterioration. At low tide, many harbor facilities are partially or completely exposed and may be inspected from piers or by boat. Taking that a step further in deeper water, divers may be utilized to provide preliminary visual information. For estimation of inspection time required for various structural elements, figure (III.8) is provided for Level I and Level II inspections.

b. Film Record

So that a diver or other inspector may bring a visual record of facilities inspected back for further analysis, a variety of still and video underwater cameras are used. Underwater 35mm photography has been in use for several decades. There are 35mm cameras made specifically for underwater use, such as the Nikonos line of Nikon. Underwater housings are also available to fit most standard 35mm "dry" cameras. Special wide angle lenses

or close up attachments are usually necessary due to the limited visibility in which most underwater inspections are performed.

In the past decade, many advancements have been made in underwater videos. The quality has greatly improved and the prices have dropped significantly. Today most inspection dive companies have underwater video capabilities. An underwater light source and wide angle lens are typically required for underwater video inspection. The video light and camera may be either surface powered or battery powered. The Navy utilizes a state of the art underwater video system known as D.U.C.T.S for Diver's Underwater Color Television System. This system incorporates the camera and light source in a single housing which has an umbilical for video signal and power to the surface.

c. Soundings

Soundings of concrete surfaces are taken by simply striking the surface with an ordinary hammer and listening to the resultant sound to identify areas of loose materials, voids, and delamination due to expansive corrosion of the reinforcing steel or to excessive bearing stresses on the member. This low tech method is very effective.

d. Rebound Hammer

The rebound hammer measures surface hardness of concrete and relates that value to the strength of the concrete. The device impacts a spring loaded hammer against the concrete surface and the size of the resulting indentation provides a measure of surface hardness. These devices must be specially fitted for underwater use, and the Navy has developed an external pressure housing for the rebound hammer which is rated to a depth of 190 ft. [57]

e. Covermeter (Rebar Locator)

The location of reinforcing steel and the thickness of concrete cover over the steel may be determined with a device known as a covermeter. Similar in some respects to a stud finder in terrestrial construction, the covermeter utilizes the magnetic characteristics of the reinforcing steel to find it in the concrete mass. The strength of the magnetic field indicates the depth of concrete cover. The effectiveness of this instrument is limited by the presence of relatively large volumes of reinforcing steel found in many massive harbor structures.

f. Corrosion Testing

One indicator of the rate of corrosion of the reinforcing steel in a marine concrete structure is the measurement of corrosion current in the steel. In relatively sound structures where the continuity of the reinforcing steel is fairly certain, such measurements can provide excellent information about the rate of corrosion. The danger in such a method for older, excessively deteriorated structures is that discontinuities in the reinforcing steel may result in extreme or erroneous local readings that are not indicative of the entire structure. Such readings may be high or low, and thus care must be taken to establish continuity throughout the area being tested before making any assessments based on data obtained.

g. Ultrasonic Methods

Ultrasonic methods of inspection may be used to detect voids and cracks not visible to the naked eye. Ultrasonic testing utilizes reflection of high frequency sound waves to map the surface and cross section of concrete. This method is basically a small area, high resolution form of sonar mapping, and more powerful and capable systems are able to map the cross section of the

concrete to certain depths, rather than just the surface. Different layers of material qualities are identified by their varying sound impedance characteristics. This method has the added advantage of being able to "see through" thin layers of soft marine growth because the growth is not dense enough to reflect a significant amount of the ultrasonic energy. An advanced ultrasonic system known as the UWATS or Underwater Acoustic Television System incorporates a 200 line scan system similar to that of a conventional TV. The object to be investigated is irradiated with ultrasonic energy from an array of transducers arranged around the camera lens, and the reflected energy is focused on the face plate of an image converter tube which converts the received signal to a visual display on a television monitor screen. This system may be used by divers or by Remotely Operated Vehicles(ROV's) [47].

h. Coring/Destructive Testing

If preliminary inspection methods reveal that levels of deterioration warrant additional destructive testing, the most common method of destructive testing involves the analysis of the properties of sections of concrete cored from the structure. Core samples may then be utilized for tests of in situ strength,

chloride levels, permeability , sulphate content, porosity, and density. High chloride levels (above 0.4% by weight of cement) are indicative of unacceptable corrosion rates[57]. The detailed effects of chlorides, sulfates and other corrosion factors are discussed in section II of this report.

3. Diving Operations

Because of the risk inherent in any diving operation, the decision to utilize divers is one that carries with it significant costs above and beyond those of normal terrestrial structural inspections.

The risk and costs of diving operations have led engineers, scientists, and constructors to rely in increasing amounts on the use of Remotely Operated Vehicles (ROV's) but the ongoing need for a man on the scene makes divers irreplaceable on many projects. Also, because this report focuses on port and harbor structures in relatively shallow, calm waters, the repair and inspection methods discussed rely heavily on diving operations.

Diving operations can be very equipment intensive, especially as operations move into colder and deeper water, and operations must be carried out in extreme environments over long periods of

time. For the purposes of port and harbor facility maintenance, the diving equipment required is relatively simple compared to that required for example to make repairs to an offshore platform in 600 feet of water. For that reason, the diving equipment presented here will be limited to simple SCUBA and Surface Supplied Lightweight diving systems. Both systems utilize standard air for breathing and both are relatively easy to support from boats or piers in a harbor environment. Figures (III.9) and (III.10) provide sketches of equipment and their general characteristics for both SCUBA, and Surface Supplied Lightweight diving apparatus.

B. ASSESSMENT

1. General

Prior to embarking on the repair of a structure, it is necessary to quantify the level of degradation of that structure. In most structural designs, a certain amount of reserve strength is available to provide a factor of safety for catastrophic loadings, additional capacity for unforeseen operational loadings, and added "stoutness" to allow for a certain amount of fatigue degradation throughout the life of the structure. Although the

terminology of this reserve strength concept is often intermingled, the fundamental question following a structural inspection is, "Do we need to repair, and if so, how much strength do we need to regain?"

For the specific case of reinforced concrete port and harbor structures, this report examines a method of engineering appraisal intended to evaluate damaged (aged) harbor structures. Evaluations are geared toward either the establishment of repair treatments which will restore structures to their original capacities, or towards the definition of new load limits for structures that cannot be economically restored.

To properly select an assessment method for any structure, the motivation for the assessment must be considered. The level of detail of the assessment will determine the cost of the assessment itself, and thus it is necessary to select an assessment method which will provide an adequate characterization of the structure's condition without making the cost of the assessment alone prohibitive. In the case of reinforced concrete port and harbor structures, there is a large quantity of ageing structures and a very small pool of available funds to effect repairs. In this instance it is neither practical nor necessary to initiate expensive and time consuming

analyses for individual structures. It is, however, quite necessary to quantify to a reasonable level of certainty both the level of structural safety of our ageing harbor infrastructure, and the present and future costs of effecting repairs to these structures. By making such quantitative analyses, it may be possible to stimulate a higher, albeit ever inadequate level of funding for repairs. At a minimum, we can more effectively prioritize the use of any available funding. The following sections of this report outline a methodology for damage/deterioration assessment.

2. Assessment Methodology

A basic methodology for structural assessment and repair is outlined in figure (III.11) and is known as the AIM (Assessment, Inspection and Maintenance) method.[31] It was brought to bear on the problem of requalifying fixed and mobile offshore oil drilling platforms and is quite applicable to other structures, including reinforced concrete port and harbor structures. As stated, the AIM objectives are:

"AIM Objectives: The basic objective of the AIM approach is to formulate an integrated, general and nonprescriptive engineering approach to the repair and requalification of

existing ageing port and harbor infrastructure. The engineer should be given the freedom to define the prescriptive procedures that can develop effective AIM programs for a specific pier or wharf."

The AIM approach further defines its three principal elements known as the AIM triangle. Assessment is defined as:

"Those engineering appraisals intended to evaluate present and future structure serviceability, and determine the desirable characteristics of present and future structure performance. This element also includes examining alternative structure maintenance programs with the objective of identifying practical candidates. These programs are focused on developing acceptable structure serviceability characteristics, while preserving essential safety, economic and environmental objectives."

Inspection is defined as:

"Those engineering and operations programs directed toward detection and documentation of defects in a structure that can lead to significant reductions in serviceability characteristics.

This element includes definition of what should be inspected, when, and how, and archiving the results for future AIM cycles."

Maintenance is defined as:

"Those engineering and operations programs developed and implemented to preserve or enable a platform to develop acceptable serviceability characteristics. This element includes consideration of a wide variety of maintenance programs intended to reduce and mitigate hazards of risks, i.e., load reductions, structure strengthening, reducing operations exposures, and increasing maintenance effectiveness."

In short, the AIM method is intended to look at a structure, assess its structural capacity in its known environment, and fix it to a level that will allow it to continue to operate commensurate with that known environment.

If we are to assess a reinforced concrete pier that has suffered from damage and deterioration, we may characterize the urgency of its repair by comparing the expected future costs associated with its failure to that of another similar structure competing for scarce repair resources. Those expected future

costs associated with failure of the structures may be calculated as follows:

$$E(Cf) = Cf\,(Pf)\,(PVF)$$

Where: $E(Cf)$ = Expected future costs due to failure

Cf = Cost of failure

Pf = Probability of failure

PVF = Present value function (equates all costs to present terms)

[35]

In figure (III.12), Bea outlines the considerations involved in what is known as a fitness for purpose evaluation. For our purposes, the hypothetical case will be the structure repaired in the case study of this report. That structure is a building supported on 21 reinforced concrete piles on the waterfront at the Naval Air Station, Alameda, California.

A fitness for purpose evaluation for this structure has the simple goal of assuring that the building may continue to serve as the workplace of the Navy's SIMA (Shore Intermediate Maintenance Activity) Divers.

As seen in figure (III.12), The fitness for purpose evaluation has Three "starting points" that all eventually lead to either a repaired/upgraded fit for purpose structure, or the decommissioning of that structure. In brief, the methodology involves comparing the known and predicted strength of the structure with the historical and predicted environmental and operational hazards. The result of this comparison is a prediction of the probability of failure of the structure. This probability of failure is then compared to the consequences of failure and if there is an acceptable relationship between the two, the structure remains in service as is. If the probability of failure is too high in relation to the cost of failure, the various repair/upgrading options must be considered. Each of these is weighed in terms of its cost versus its effect on the probability of failure and a repair procedure is chosen which will reduce the probability of failure to a level commensurate with the cost of failure.

Two facts are important to note here. First is that there is an optimal relationship that exists between cost of failure and probability of failure as shown in figure (III.13). This applies to both initial design and construction costs and to the costs associated with various repair methods considered.

Second, the repair of a structure may include re-definition of loads to more accurately reflect current information about the severity of operational and environmental loads to be expected. In many current cases, reevaluation of loadings has shown that original criteria have been over conservative. Conversely, there are many cases where original criteria are superseded by more severe current criteria and predicted environments.

From the above, it is clear that there are two key numbers in the requalification picture. One is the cost of failure and repairs or rehabilitation, and the other is probability of failure. For any structure then, the task of reducing costs associated with failure is centered on reducing the cost of failure and/or reducing the probability of failure. Both are examined further below.

C. Reducing the Cost of Failure

For a reinforced concrete pier, costs of failure will include:

- -- the cost of the structure
- -- damage to moored ships
- -- loss of life
- -- damage to the environment

-- loss of operations

In order to reduce the cost of failure "by decree", it may simply be necessary to limit the aforementioned costs by reducing traffic (ships, men, and equipment) on to pier to minimum levels, thus reducing their exposure to failure. In the extreme case, it may be necessary to close down operations completely, thus limiting the cost of failure to the cost of the structure and its effects on the environment. This method, however, also limits in the extreme the operational utility of the pier. In general terms, steps taken to reduce the cost of failure of a structure "by decree", will limit the operational utility of the structure as well, unless those steps (at some cost) are designed to actively change the results of a catastrophic failure of the structure.

For an offshore oil platform, an example of this would be the installation of a subsea blowout preventer (BOP) which in the event of failure is designed to close off the well opening to prevent a massive influx of oil into the surrounding waters. For a concrete pier, such measures typically encompass backfitting the pier utility systems with current design safety features to limit the effects of steam, oil, and electrical distribution systems on the environment and personnel.

For the pier structure itself, efforts to reduce the cost of failure are typically limited to the "by decree" methods listed above. Any other actions which reduce the cost of failure are generally coincident with structural improvements which are intended to improve capacity and reduce the probability of failure. These methods are described in the following section.

D. Reducing the Probability of Failure

The probability of failure of a structure as it relates to the expected future costs associated with failure is generally broken into two parts:

$$Pf = Pfo + Pfs$$

Where: Pfo = Probability of failure due to operations

Pfs = Probability of failure of the structure

When assessing a pier for repairs or reclassification of its operating capacity, reducing the probability of failure due to operations is similar to reducing the cost of failure due to operations. That is, one may lower the operating load limits on the pier, but will suffer a commensurate decrease in utility of the pier. For example, a pier may be limited to smaller ships, trucks, cranes, etc. to reduce to potential for failure, but if it

is necessary to operate the pier at its original load limits, then these measures may only be temporary until structural repairs can allow for a return to original design capacities. Other ways to reduce the probability of failure due to operations are:

-- safety training programs for crane operators

-- establishment of requirements for tugs to reduce berthing loads

-- scheduling and planning high load operations for off peak hours, or limiting them to specific areas where pier capacity remains good

These methods are also very applicable and recommended even for undamaged piers, but are often times dictated by deterioration.

The reduction of the probability of failure of the pier structure is where repairs play a major role. If the structure has been damaged or deteriorated to some capacity below its original design (intact) capacity, then it is necessary to assess its reduced (damage) capacity to determine a logical scope of

repair and maintenance to allow for the continued operation of the structure.

One can see from the foregoing that in order to reduce the probability of failure, one must alter the factors affecting the probability of failure. That may be accomplished for a deteriorated reinforced concrete pier in any one of the following ways:

Reduction in loading uncertainty - gathering data throughout the life of the structure to better define the range of loadings to be expected.

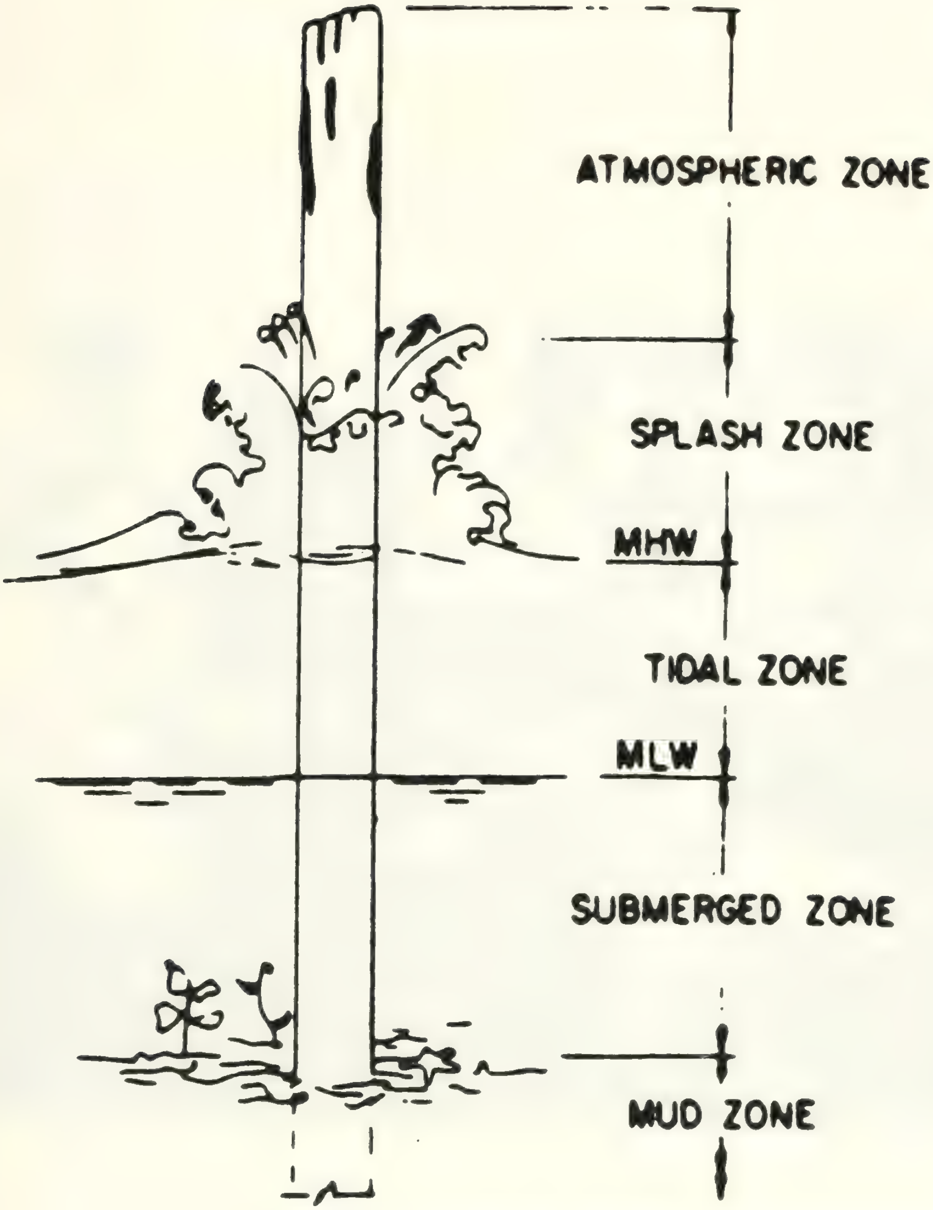
Reduction in capacity uncertainty - gathering data throughout the life of the structure to document based on actual loads the capacity of the structure.

Reduction in the median load - "by decree" as described above, which reduces the utility of the pier, or by actual data which supports a lower median load.

increase in the median resistance - by augmenting or repairing the pier structure (comparing the net gain for various repair methods).

F. Conclusion

Unfortunately, the current state of most of the maritime infrastructure in the United States dictates that ageing facilities stand in line for scarce funding so that only the most critical facilities (if any) receive attention. This has created a de facto system of treating only gross safety deficiencies with limited available funds. As an example, the U.S. Navy utilizes a system wherein every naval base prepares a report of the condition of each one of its facilities (including port and harbor facilities). Each facility is assigned a condition code and typically only a handful of facilities out of hundreds get significant funding for repair projects. It is thus necessary that with the limited resources available we are able to logically prioritize to assure that we are eliminating our greatest risks first. Application of the assessment methods described herein is an excellent start towards establishing accurate priorities.



Classification of marine structure exposure zones.

Figure III. 1 Classification of Marine Structure Exposure Zones

Figure III.2 Diver Engaged in Underwater Visual Inspection [47]

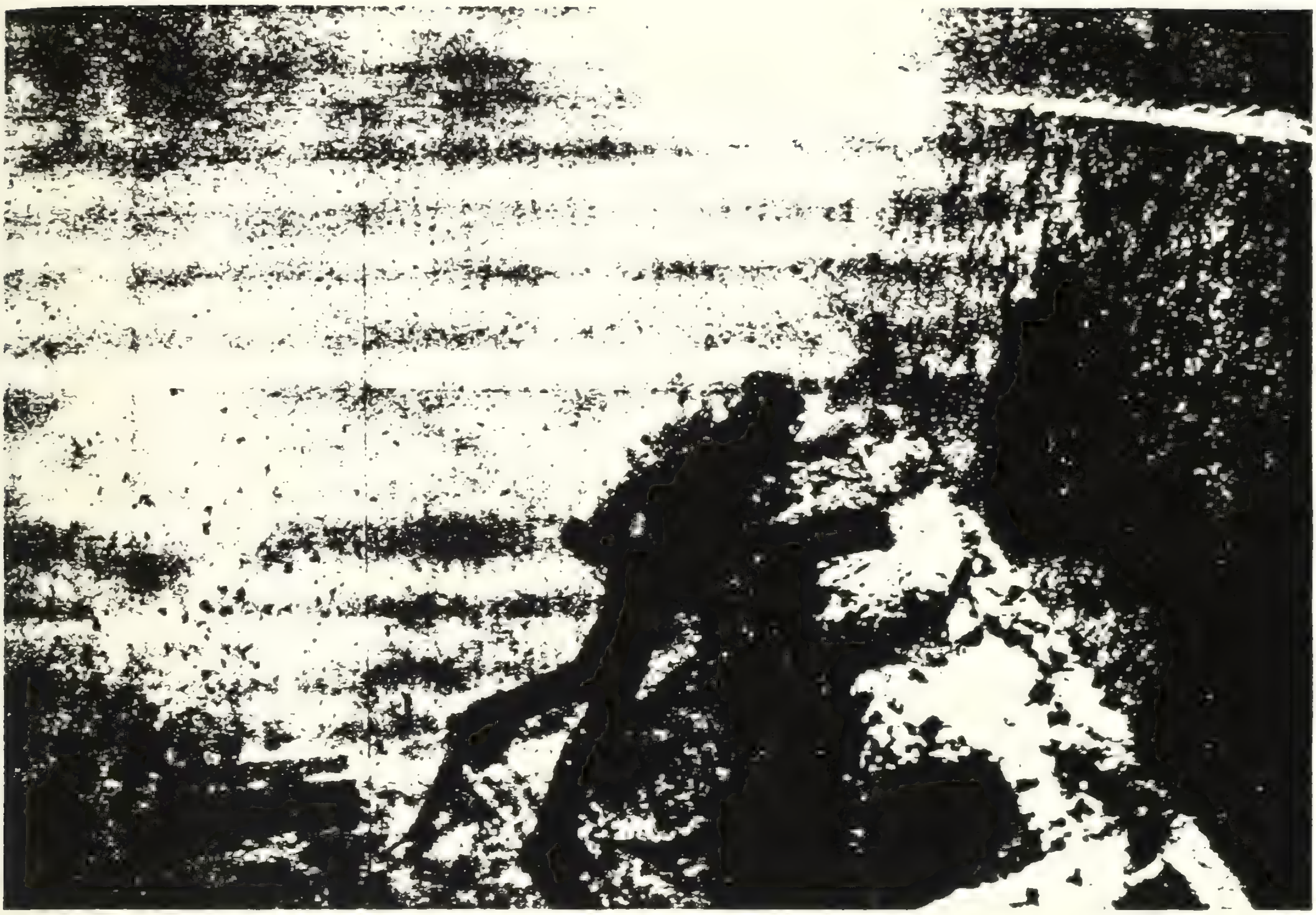


Figure III.3 Diver Engaged in Underwater Inspection

PILE INSPECTION RECORD

LOCATION		DATE	DIVERS
PIER NAME/NO		PILE TYPE ☐ BEARING ☐ FENDER ☐ SHEET	PILE MATERIAL ☐ TIMBER ☐ STEEL ☐ REINFORCED CONCRETE
WATER DEPTH	TIME OF DAY	TIDE	DEPTH OF DAMAGE FROM DATUM=GAUGE DEPTH-TIDE

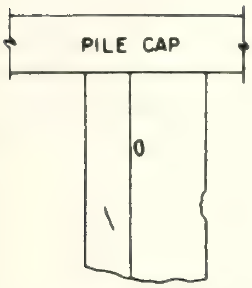
BENT NO	PILE NO	NI	PILE CONDITION					TYPE DAMAGE			GAUGE DEPTH DAMAGE	DIMENSIONS OF DAMAGE			COMMENTS
			ND	MN	MD	MJ	SV	MECH	BIO	FUNC		HGT	WIDTH	PENETR	

Figure III.4 Typical Inspection Report Format

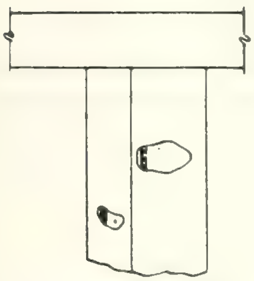
CONCRETE PILE CONDITION RATING — EXPLANATION

NI — NOT INSPECTED, INACCESSIBLE OR PASSED BY

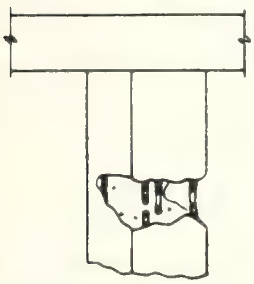
ND — NO DEFECTS:
- fine cracks
- good original surface, hard material, sound



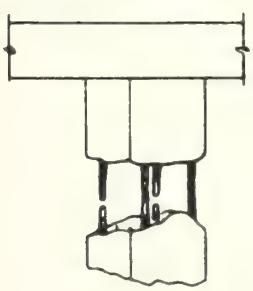
MN — MINOR DEFECTS:
- good original section
- minor cracks or pits
- surface spalling that exposes course aggregate
- small chips or popouts due to impact
- slight rust stains
- no exposed re-bar
- hard material, sound



MD — MODERATE DEFECTS:
- spalling of concrete
- minor corrosion of exposed re-bar
- rust stains along re-bar with or without visible cracking
- softening of concrete due to chemical attack
- surface disintegration to one inch due to weathering or abrasion
- reinforcing steel ties exposed
- popouts or impact damage



MJ — MAJOR DEFECTS:
- loss of concrete (10-15%)
- one or two re-bars badly corroded
- one or two ties badly corroded
- large spalls six inches or more in width or length
- deep wide cracks along re-bar
- dummy areas full width of face



SV — SEVERE DEFECTS:
- two or three re-bars completely corroded
- no remaining structural strength
- significant deformation

Explanation of pile condition ratings for concrete piles.

Figure III.5 Concrete Pile Condition Ratings [61]

Level	Purpose	Detectable Defects		
		Steel	Concrete	Wood
I	General visual to confirm as-built condition and detect severe damage	Excessive corrosion Severe mechanical damage	Severe mechanical or ice (freeze/thaw) damage	Severe damage due to marine borers Mechanical over-load resulting in broken piles Severe abrasion
II	Detect surface defects normally obscured by marine growth	Moderate mechanical damage Corrosion pitting	Surface cracking due to mechanical overload Deterioration of concrete due to sulphate action, moderate freeze/thaw damage, etc. Severe corrosion of rebar Spalling of concrete surface	External damage due to marine borers Splintering due to mechanical overload
III	Detect hidden and beginning damage	Thickness of material	Location of rebar Beginning corrosion of rebar Internal voids Change in material strength	Internal damage due to marine borers (internal voids) Decrease in material strength

Figure III.6 Summary of Detectable Defects by Level of Inspection [61]

Method	Material	Defects	Advantages	Limitations	Remarks
Visual	All materials	Surface cracks/pitting, Impact Damage, Surface Corrosion, Marine Fouling, Debris, Scouring, Concrete spalling/crumbling	Results easy to interpret. Can be conducted with a variety of techniques.	Limited to surface defects. Surface must be cleaned for detailed observation.	
Magnetic Particle	Magnetic materials only	Surface cracks, laps, seams, pits and some near-surface flaws.	Easy to interpret.	Thorough cleaning required. Weather dependent in splash zone. Limited to surface and near surface defects. Does not measure depth of defect. Interpretation done only <u>in situ</u>. Present equipment limited to diver use. Magnetic materials only. No permanent record. Cumbersome to perform underwater.	Surface Support required.
Magnetographic Method	Magnetic materials only	Surface cracks, laps, seams, pits and some near surface flaws.	Simple to perform. Permanent record. Signal enhancement possible. Defect depth can be obtained. Interpretation conducted on surface. No diver NDT qualifications required.	Thorough cleaning required Limited to surface and near-surface defects. Geometry of structure can be prohibitive. Magnetic materials only. Equipment limited to diver use.	Potential for application by mechanical manipulators
Fe Depth Meter	Reinforced concrete	Depth of steel reinforcement in concrete.	Easy to perform. Results immediate. Can be preformed by mechanical manipulation.	Thorough cleaning required Bar size must be known for greatest accuracy. No data recording feature.	
Ultrasonics	Metals, Concrete, Plastics, Creamics, Glass, Rubber Graphite	Cracks, Inclusions Porosity, Laminations, Bursts, Grain size, Lack of bond, Lack of weld penetration and fusion, Thickness variations.	High sensitivity. Fast Penetrates up to 10mm of steel. Accurate flaw location. Access to only one side needed.	Thorough cleaning required. Operator skill is required. Usually no permanent record. Comparative standards only Surface roughness can affect test. Difficulty with complex shapes. Present equipment limited to diver use.	
Acoustic Holography	Same as above	Same as above.	Provides three-dimensional view of internal defects which can be precisely measured and located.	Thorough cleaning required. No field experience underwater. Present equipment limited to diver use.	
Radiography	All materials	Internal defects such as inclusions, porosity, shrinks, corrosion, lack of penetration and fusion in welds. Thickness measurements.	Provides permanent record Standards established. Accepted by codes and and industry. Portable.	Thorough cleaning required. Potential health hazard. Defect must be at least 2% of total section thickness. Difficulty with complex geometry. Water must be displaced between source and subject. Requires access to both sides. Present equipment limited to diver use.	
Corrosion Potential	Metals	Tests cathodic protection system by measuring interface potential between structure and seawater.	Simple to perform. Rapid measurements. Easy to interpret. Can be performed by mechanical manipulation.	Thorough cleaning required. Measures external potential only.	

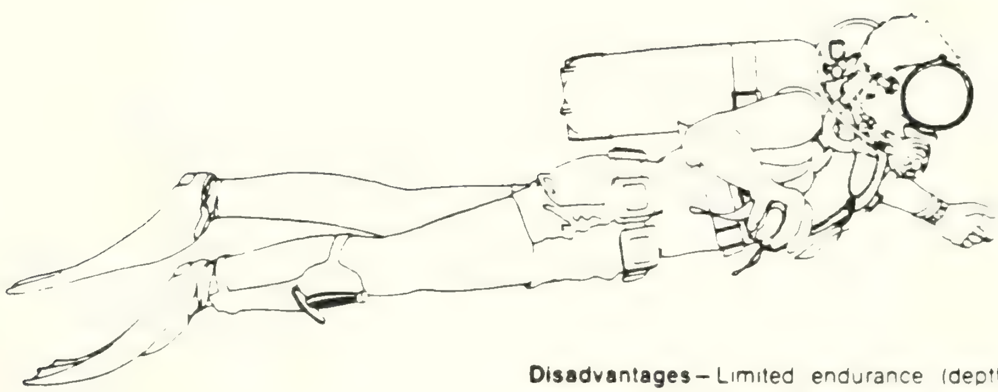
Figure III.7 Current Underwater Non-Destructive Testing Techniques, Advantages and Limitations [54]

Structural Element	Inspection Time Per Structural Element	
	Level I (min)	Level II (min)
12-in. steel H-pile	5	30
12-in.-wide strip of steel sheet pile	3	15
12-in. square concrete pile	4	25
12-in.-wide strip of concrete sheet pile	3	15
12-in.-diam timber pile	4	20
12-in.-wide strip of timber sheet pile	3	15

Figure III.8 Typical Underwater Inspection Times for Various Structural Elements [61]

SCUBA

GENERAL CHARACTERISTICS



Minimum Equipment– Open-circuit SCUBA
Life preserver
Weight belt
Knife
Face mask
Swim fins

Principle Applications– Shallow water search
Inspection
Light repair and recovery
Clandestine operations

Advantages– Rapid deployment
Portability
Minimum support
Excellent horizontal and vertical mobility
Minimum bottom disturbances

Disadvantages– Limited endurance (depth and duration)
Breathing resistance
Limited physical protection
Influenced by current
Lack of voice communication

Restrictions– Working limits–
Normal 60 feet/60 minutes
Maximum 130 feet/10 minutes
Current–1 knot maximum
Diving team–minimum 4 men

Operational Considerations– Buddy and standby diver required
Small boat required for diver recovery
Avoid use in areas of coral and jagged rock
Moderate to good visibility preferred
Ability to free ascend to surface required

Figure III.9 SCUBA General Characteristics [47]

LIGHTWEIGHT DIVING

GENERAL CHARACTERISTICS

Minimum Equipment –	Diver's Mask USN MK 1 or Jack Browne mask Wet suit Weight belt Knife Swimfins or shoes Surface umbilical
Principle Applications –	Shallow water search Inspection and major ship repair Light salvage
Advantages –	Unlimited by air supply Good horizontal mobility Voice and/or line pull communications Fast deployment
Disadvantages –	Limited physical protection Limited vertical mobility Large support craft required
Restrictions –	Work limits – Jack Browne – Normal 60 feet/60 minutes Maximum 90 feet/30 minutes Work limit – MK 1 without come home bottle Maximum 60 feet Work limit – MK 1 without open bell Maximum 130 feet/10 minutes Work limit – MK 1 with open bell Maximum 190 feet/60 minutes Current – 2.5 knots max
Operational Considerations –	Ability to free ascend to surface required, with exception noted in Para 6.8.7.2 Adequate air supply system Standby diver required

U S NAVY DIVING MANUAL

Figure III.10 Lightweight Diving System

General Characteristics [47]

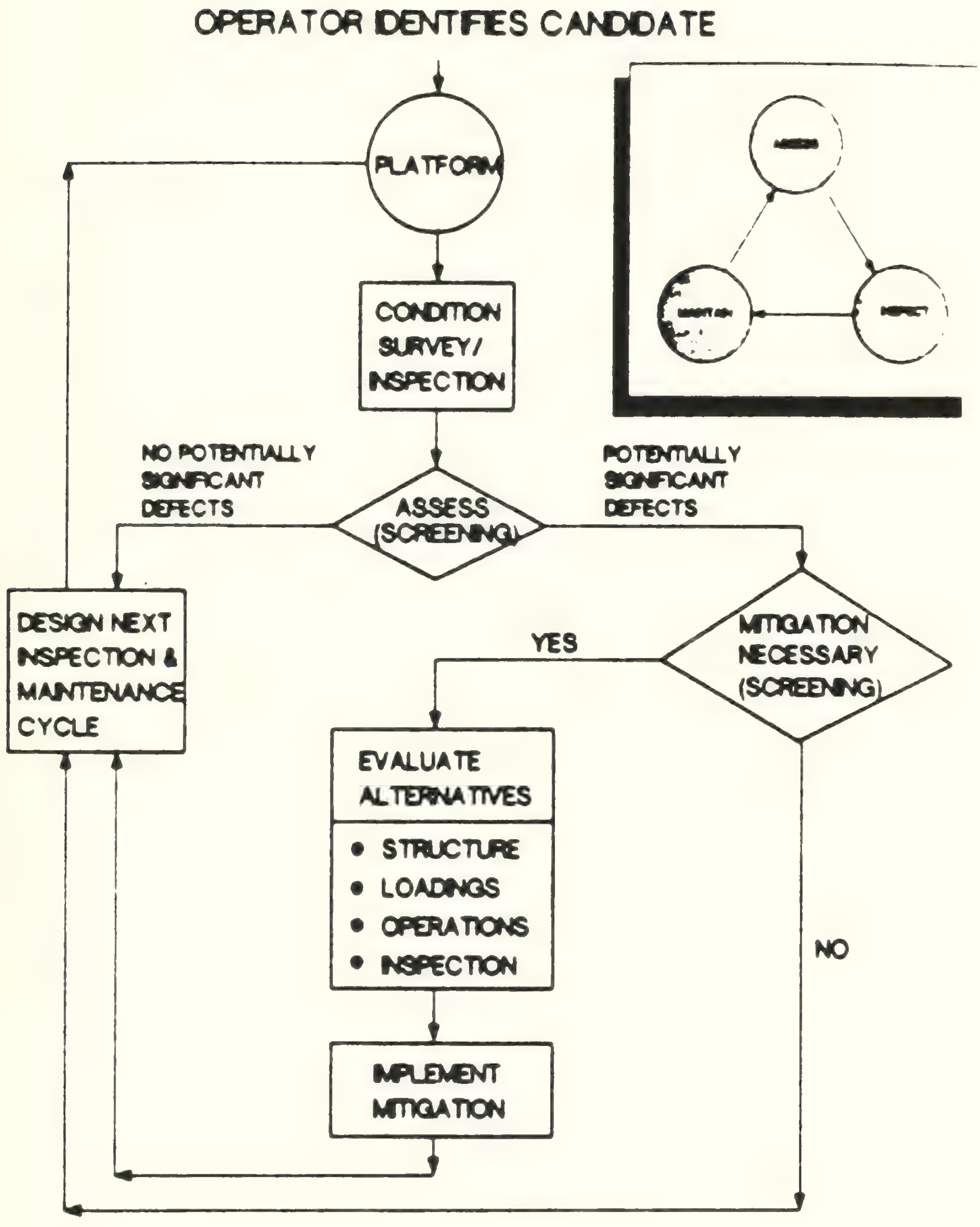


Figure III.11 Assessment, Inspection, Maintenance Triangle [31]

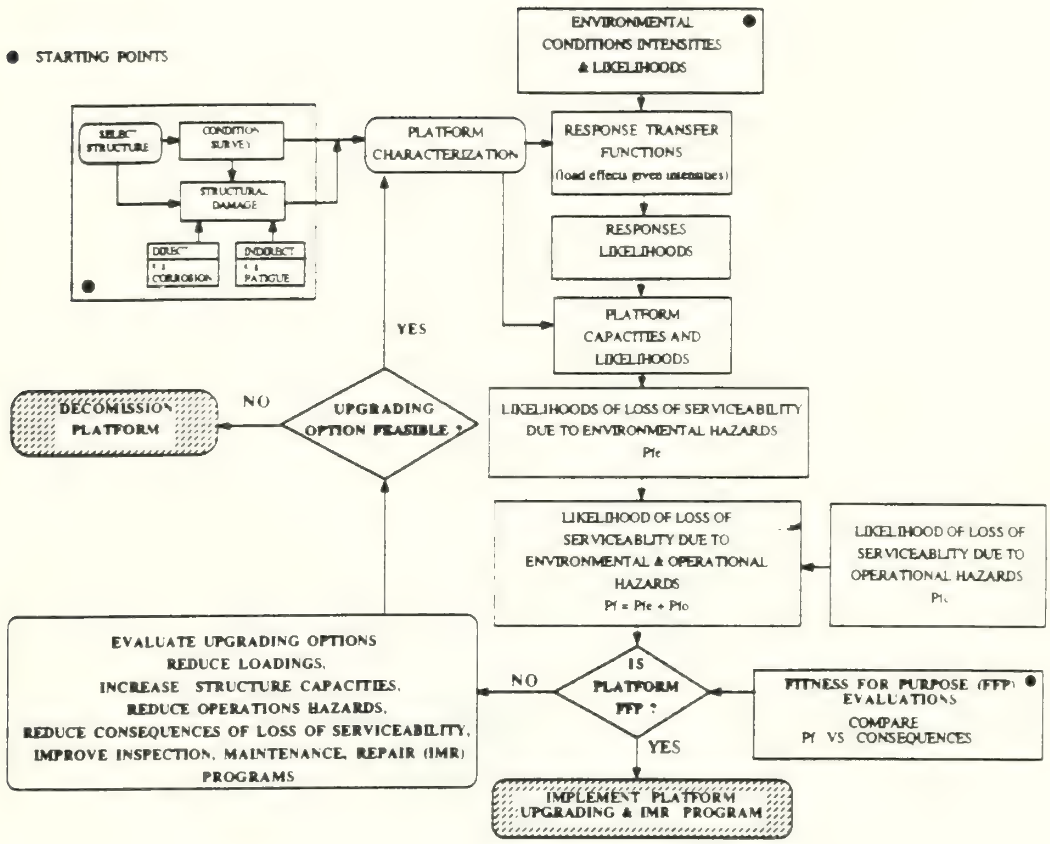


Figure III.12 Ocean Structure Requalification

Logic Diagram [Bea]

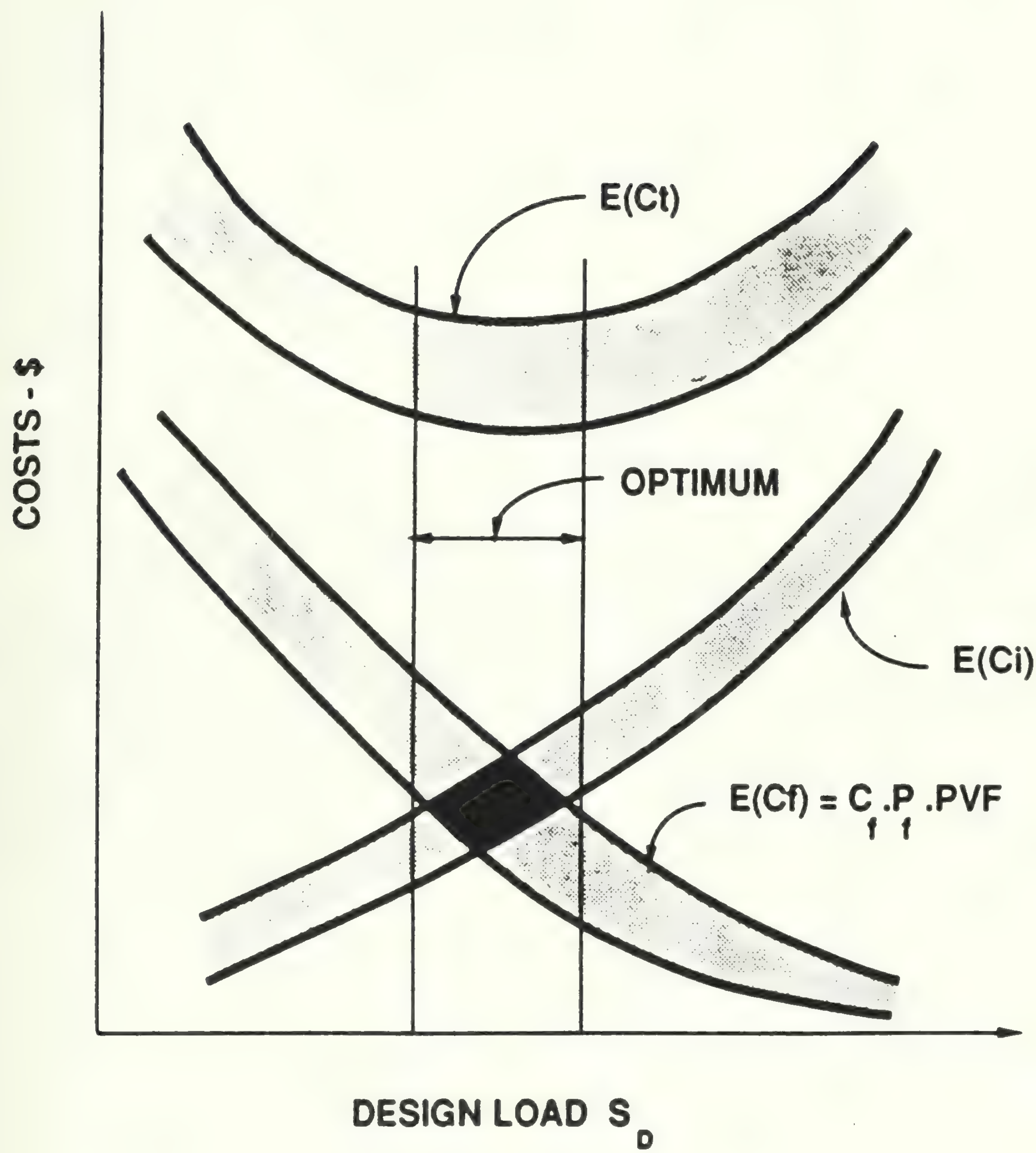


Figure III.13 Optimization of Total Initial and Future Costs

SECTION IV

REPAIR TECHNIQUES FOR CONCRETE PORT AND HARBOR STRUCTURES

by

Jim Schofield

A. Introduction

High quality repairs to concrete port and harbor structures may be achieved through application of some fundamentals which have been recognized for many years. High quality materials applied to a well prepared surface using carefully considered and executed methods will result in a good repair which has the required strength, durability, appearance and economy. These basic requirements are summarized in figure (IV.1).

In principle, these fundamentals are very simple and easy to discuss while sitting high and dry in the classroom or office. In practice, putting these simple tenets into effect has occupied the efforts and imaginations of many engineers, constructors, and scientists over the past 50 years. The following sections summarize the fundamental elements of the repair process for concrete port and harbor structures, and highlights some specific cases and examples for the purpose of illustration of the methods discussed.

The repair process is divided into three areas. First the preparation of the surface to be repaired. Second the treatment of reinforcing steel, and finally the placement of new material. Although a wide variety of damage and repair

scenarios exist, this division of the process is representative of most concrete repair requirements encountered in the water. Figure (IV.2) provides a summary illustration of the types of damage to be repaired on a typical concrete harbor structure in sea water. These damage modes include:

-- cracking due to corrosion of reinforcing steel

--cracking due to freezing and thawing

--physical abrasion due to freezing and thawing

-- chemical decomposition of hydrated cement

-- early thermal contraction cracking

--drying shrinkage

--plastic settlement

B. Surface Preparation

1. Introduction

The execution of quality repairs to reinforced concrete marine structures is similar to repainting a house in at least one critical way. That is, to effect quality repairs, one must properly prepare the surface of the area to be repaired, or the results will not be very durable. As with house paint, a quick job on a poorly prepared surface may be inexpensive and produce good short term cosmetic results, but the underlying structural

preservative value of the effort is minimal, if not counterproductive.

The surface preparation of marine concrete prior to repair may be classified in two ways. First is the removal of deteriorated concrete. This includes the exposure or replacement of sections of reinforcing steel, discussed in section IV. Second is the cleaning of the surface to be repaired, which includes the removal of contaminants such as oil, dirt, and marine growth. Particular attention is given in section I (Characterization of the Marine Environment) to the rapid development of marine growth and its negative effects on proper bonding of repair materials.

2. Removal of Deteriorated Concrete

Typically, a repair or rehabilitation project for marine concrete will involve the removal of some deteriorated concrete. The effectiveness of various removal techniques may vary for deteriorated and sound concrete, and in some cases, ongoing and uninterruptible structural capacity requirements may prohibit the removal of concrete beyond a certain level. Similarly, selection of a proper removal technique may have a significant effect on the length of time that a structure must be out of service. Of

particular importance in marine concrete, the same removal techniques may not be suited for all portions of a given structure at any given time. A classic example of this is the varying conditions (primarily wetting and drying) encountered in the tidal or splash zone of marine structures. In some cases it may be beneficial to schedule portions of work to coincide with the period of low tide. Additionally, selection of a removal method is highly dependent on the magnitude of the repair effort.

Concrete removal methods may be classified [41] by the way in which the process acts on the concrete:

Blasting Methods - Blasting methods generally employ rapidly expanding gas confined within a series of boreholes to produce controlled fracture and removal of the concrete. Explosive blasting involves placement of explosives in boreholes and detonating the explosive. High pressure carbon dioxide blasting utilizes high pressure carbon dioxide gas to break down masses of materials. Blasting methods are obviously only applicable on very large scale demolition/repairs, and in areas where impact on the environment is minimal.

Cutting methods - These include mechanical sawing, high pressure water jets or intense heat to cut around the perimeter of concrete sections to allow for their removal. Of these three methods, high pressure water jets are most applicable to marine concrete due to their relative simplicity and the immediate availability of the cutting medium (water). Water jets are also discussed later in their less concentrated applications for final cleaning.

Impacting methods - Impacting methods involve striking damaged concrete with anything from a wrecking ball to a chipping hammer. Specific applications vary with the mass of concrete to be removed.

Presplitting methods - These employ mechanical devices (wedges), water pressure pulses, or expansive chemicals inserted into existing cracks or boreholes drilled at points along a predetermined line to induce a crack plane which allows for removal of concrete.

3. Cleaning of the Concrete Surface

Following the macro level removal of deteriorated or damaged concrete, it is necessary to prepare the repair surface on the

micro level. In many cases, depending on the scope of the repair effort, the volume of concrete in question, and the level of deterioration or damage, the removal of deteriorated concrete and the preparation of the surface may be accomplished in a single step. For our purposes, surface preparation is treated separately and may be considered low intensity or small volume surface preparation as opposed to the more intrusive methods described in the previous section. These methods may be categorized into four types of cleaning:

Acid etching - This method is obviously limited to repairs above the waterline and has limited application in this era of increasing emphasis on environmental impact.

Hydroblasting/Water Jetting - This method is most applicable to marine projects as mentioned above, and is especially useful for "last minute" cleaning of repair surfaces prior to placement of repair materials. A high velocity water jet (preferably from a reactionless nozzle) is used to blast the surface free of marine growth and other fouling.

Mechanical Abrasion - Either hydraulic or pneumatic abrading devices in the form of chipping hammers, chain-hammer whips, scabblers, and the like are most effective for cleaning the

repair surface and providing a roughened face for proper bonding and keying.

Sandblasting - Applicable only above the waterline, and increasingly eschewed due to environmental considerations and cost.

Figure (IV.3) provides a table of marine growth removal rates from concrete structures for various methods of cleaning. These rates are most applicable to cleaning prior to inspection, but are indicative of relative rates between the various methods.

As with many other aspects of concrete engineering, proper surface preparation is heavily dependent on quality control of both the design and execution of repairs. A hasty job will not achieve the desired results, and in many cases the level of deterioration of the concrete makes it more practical to completely replace a concrete member. Where repairs are to be effected, complete and timely surface preparation as described above must be specified and enforced if the repairs are to have more than a cosmetic, short term value.

C. Treatment of Reinforcing Steel

1. Cleaning

In addition to the removal of contaminated and deteriorated concrete, it is necessary in the repair of marine concrete to consider the preparation and treatment of the reinforcing steel. First, concrete should be removed to a depth that allows treatment of exposed reinforcement for its entire circumference to allow for placement of repair material behind the reinforcement.

Once properly exposed, it is necessary to remove corrosive products from reinforcement and, to the maximum extent possible, to achieve a rust free surface. In the marine environment, this is rarely completely possible, but it is best attempted with water jetting methods. The reinforcing must also be exposed around its entire circumference to insure that when new material is placed, it achieves good bond on the steel. The reinforcing can also be coated to protect it against further corrosion, but this procedure is also limited under water, and most typically the only coating placed on the reinforcing steel is the ultimate repair material which fills the voids and completely covers the steel.

2. Zinc Spraying

An innovative method for corrosion control of reinforcing steel involves the spraying of zinc over exposed, cleaned reinforcing steel and surrounding external concrete surface. The method is intended to be a low cost alternative to conventional impressed current cathodic protection systems.
The procedure creates reliable electrical contact with the steel, and provides a large contact area between the anode (zinc coating) and the concrete. In the marine environment, surrounding humidity typically results in significant concrete conductivity which permits good current for cathodic protection. Please see Appendix 5 for more detailed discussion of cathodic protection systems.

Limited testing by the Florida Department of Transportation has been encouraging but as yet inconclusive. Further testing of this method is underway by the Florida Department of Transportation and the California Department of Transportation to determine the effectiveness of the current levels generated, the effect of service and application parameters on the durability of the sprayed zinc, and the effectiveness of the method under field conditions [63].

3. Replacement/Addition

A very good solution to the problem of corroded and exposed reinforcing is the replacement of the steel, or addition of additional steel. Normal lapping lengths may be used. Figure (IV.4) illustrates for example purposes lap splice lengths for grade 60 reinforcing bars. Splice lengths depend on bar size and concrete strength.

D. New Material Selection

1. Introduction

Materials used for repair of marine concrete for port and harbor structures may be classified in four fundamental groups: Cementitious, epoxy resin, polyester resin, and polymer modified cementitious systems. Each type of material has general characteristics that make them desirable for different applications. Their selection is based on the properties that are required in the end product such as compressive strength, modulus of elasticity, flexural strength, tensile strength,

thermal expansion properties, permeability, service temperatures, pumpability and strength development rate.

Most typically, it is desirable to select a repair material that has properties as closely equalling the properties of the original material as possible. That is ideal, however, and it is quite often necessary to weigh several conflicting factors in the selection of a material. For example, for a relatively straightforward concrete encasement of an abraded concrete pile, it is desirable to use a cementitious grout whose modulus of elasticity and linear coefficient of thermal expansion matches that of the existing concrete. In this way, the potential for delamination of the repair material under varying conditions of stress and temperature fluctuation is reduced. In practice, however, it is virtually impossible to design and mix cementitious grout to a degree of accuracy required to match its modulus of elasticity to that of existing (perhaps 50 year old) concrete.

The following are definitions of the four material groups identified above:

Cementitious Grout - "a mixture of cementitious material and water, with or without aggregate, proportioned to produce a pourable consistency without segregation of the constituents" Grout material selection must be driven by the type of repair

being performed and in particular the need for the repair material's:

-- strength

-- permeability

-- pumpability

[50]

Epoxy Resin - a class of organic chemical bonding systems used in the preparation of special coatings or adhesives for concrete or as binders in epoxy resin mortars and concretes

Polyester Resin - one of a large group of synthetic resins, mainly produced by reaction of dibasic acids with dihydroxy alcohols; commonly prepared for application by mixing with a vinyl-group monomer and free radical catalysts at ambient temperatures and used as binders for resin mortars and concretes.

Polymer modified Cementitious systems - concrete in which an organic polymer serves as the binder; also known as resin concrete.

Figure (IV.5) provides a tabulated summary of the general property parameters of the materials above.

E. New Material Placement

1. Concrete Encasement

A common repair method for concrete piles is to encase the damaged or deteriorated section of the pile in new concrete, after appropriate surface preparation as described above. Both flexible and rigid forms are used extensively for this purpose. The area encased is most often defined by the limits of the splash zone, and the concept is a simple one. Figure (IV.6.1) lists planning and estimating data for concrete pile repair using concrete encasement. Figures(IV.7), (IV.8) and (IV.9) are examples of forming methods utilized for concrete encasement. Figure (IV.10) lists a generic repair procedure for concrete encasement. Figures (IV.11) through (IV.13) are taken from the State of Florida Bridge Repair Manual and provide additional summaries of concrete encasement methods and procedures. Note that throughout the various procedural lists, the common themes of surface preparation, steel treatment, and new material selection and placement prevail.

2. Epoxy Injection

Small to medium cracks caused by manufacturing, installation, weathering , deterioration, or reinforcing steel corrosion allow water to penetrate concrete harbor structures. Repairs for these cracks may be effected by filling and sealing them by injecting a low viscosity epoxy resin and sealing the outside with epoxy paste. Routing and cleaning of cracks are performed with conventional hand and power tools. Injection of the epoxy for smaller jobs can be done with a hand operated caulking gun. Figure (IV.14) illustrates the use of the hand operated caulking gun. Larger jobs are usually done with special epoxy pumps, operating at less than 150 psi, with mixing tank, injection hose, and controls. Figures (IV.15) and (IV.16) diagram injection and patching techniques for damaged concrete. Figure (IV.17) outlines planning and estimating data for repair by epoxy injection. Figure (IV.18) outlines a repair procedure for the utilization of epoxy adhesive. Of special note in figure (IV.18) is that typical epoxy grout will not set in temperatures below 40 degrees Fahrenheit. Conversely, when ambient temperatures are high (above 80 degrees F), epoxy grout may "flash set" almost instantaneously.

3. Patching of Spalled Concrete

In areas where expansive corrosion or excessive bearing stresses have resulted in spalling of concrete from slabs or piles, it is common to mortar patch the spalled areas. Either cementitious mortar or epoxy mortar may be used, but in any case, development of a good bonding surface for the patch is essential to prevent continued spalling. Figures (IV.19) and (IV.20) diagram placement of patches and figure (IV.21) lists a generic procedure for patching using either cementitious material or epoxy with cement and sand fillers. Figure (IV.22) gives planning and estimating data for concrete pile repair using epoxy patching.

4. Pneumatic Projection of Concrete (Shotcrete)

For the repair of relatively large areas with a relatively thin layer of material (as in the case of pier decking) the application of pneumatically injected concrete (shotcrete) is very effective. It affords new cover for existing reinforcing steel without the time and labor intensive effort required to put forms in place. A generic procedure for shotcrete repairs is outlined in figure (IV.23). Figure (IV.24) is an example diagram for the placement of shotcrete.

5. Concrete Pumping

Of special interest in the repair of concrete port and harbor structures is the art of pumping concrete. Because of there ability to provide large quantities of concrete to inaccessible locations, concrete pumps are increasingly common on job sites. They are especially useful for underwater work because of their built in ability to tremie concrete into the bottom of a form underwater. Figures (IV.25) and (IV.26) demonstrate the use of concrete pumps for underwater placement of concrete. In figure (IV.26), the divers are communicating with the pump operator while they accurately direct the concrete placement. A summary of on site concrete pumping hints is provided in figure (IV.27).

F. Effectiveness of Repairs

1. Ten Golden Rules

To insure that repair efforts on concrete port and harbor structures are long lasting and effective, one may begin by reviewing proper procedures for the design and placement of new concrete. These are very well summarized in the "Ten Golden Rules For Placement of Concrete in the North Sea" as follows:

1. Select high quality materials

2. Get the mix proportions right
3. Employ modern automatic batching plants
4. Develop sound work procedures beforehand
5. Compact the concrete generously
6. Ensure adequate cover to rebars
7. Pay attention to construction joints
8. Make allowances for temperature
9. Keep design simple
10. Use trained and skilled operators

[3]

Although those rules were written for the case of large volume concrete pours on North Sea oil platforms, they apply on a smaller scale to harbor structure repair. In addition to following these rules for good quality repairs, there are methods discussed below to reduce rates of corrosion and deterioration either before or after repairs are effected.

2. Cathodic Protection

Cathodic protection is based on the principle of inhibiting corrosion by passing an electric current onto the steel reinforcement, effectively making it the cathode in a corrosion cell , where the "sacrificial" anode is consumed, protecting the

reinforcing steel. Appendix 5 provides a detailed summary of cathodic protection theory and design considerations.

3. Chloride Extraction/Sealing

Chloride extraction is a means to improve the durability of repaired or sealed concrete surfaces by reducing the chloride content in the repaired layer prior to the placement of new or cover material. This method requires the application of a current to reinforcing steel, and placement of a temporary anode layer on the surface of the concrete. After a matter of weeks, enough of the chloride ions present in the concrete migrate to the anode layer to allow stripping of the anode layer and subsequent cover of the low chloride concrete. Appendix 5 also outlines the theory and application of chloride extraction, as well as a cost comparison between cathodic protection and chloride extraction.

Following the chloride extraction process, repair materials are placed to prevent further intrusion. Those repair materials must have a low permeability to insure against a repeat of the original corrosion problem. Many types of coatings are available, but if high strength, low permeability concrete is used with adequate cover of any exposed reinforcing steel, additional

coatings should be unnecessary. For information purposes, a brief summary of several types of coatings is provided here, commencing with epoxies, which are effectively the only surface sealant which can be used below the water line or in the splash zone.

Epoxies - normally the surface seal type, have a good record for resistance to chloride, carbon dioxide and water penetration. They do not resist ultraviolet light well but are otherwise very durable. Bonding characteristics are excellent..

Bitumen - good bonding properties and excellent resistance to carbon dioxide, chloride and water. They are extremely durable with a life expectancy of over 20 years, as long as they are not exposed to ultraviolet light. Bitumens may be applied to slightly damp surfaces but this reduces their bonding capacity and durability.

Silane and Siloxane - surface penetrating materials, to depths up to 4mm. Excellent water repellants, but only effective when applied to very dry surfaces.

Acrylics - good resistance to UV light, and good bonding properties. When they are provided in emulsion form, they can bond to damp surfaces, but they have limited resistance to chloride and water penetration. [42]

Figure (IV.28) provides further information about the degradation rates of these various coatings.

Figure (IV.29) summarizes types of coatings, including application layer information and typical membrane thickness.

G. Conclusion

The durability of concrete repairs is obviously not only a function of the repair procedure. Only the combination of high quality materials, good workmanship, and a well designed procedure will prevail in the face of the marine environment and ever increasing operational requirements.

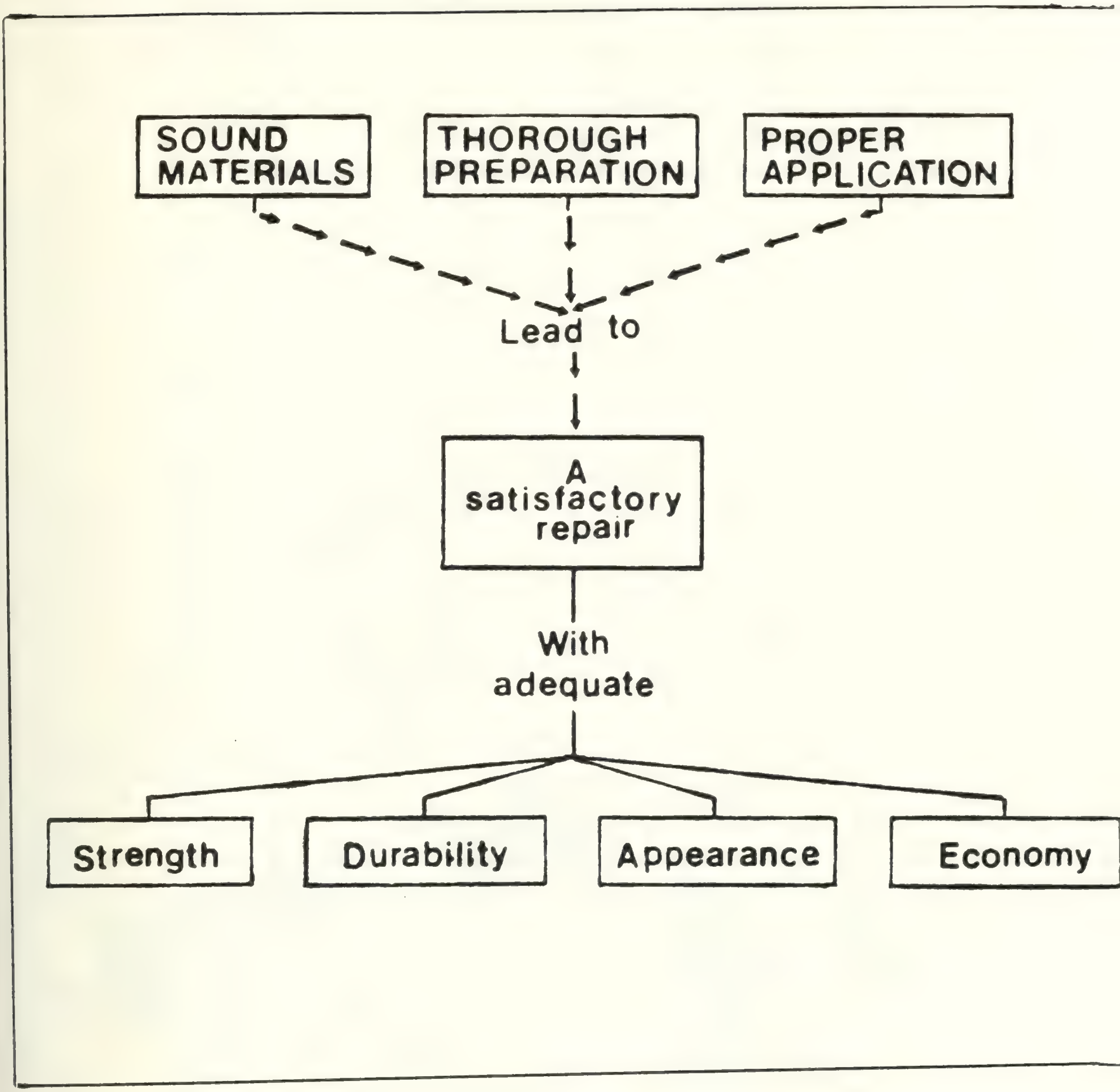


FIGURE IV.1 Basic Repair Considerations

Deterioration of Concrete Structures in Sea Water

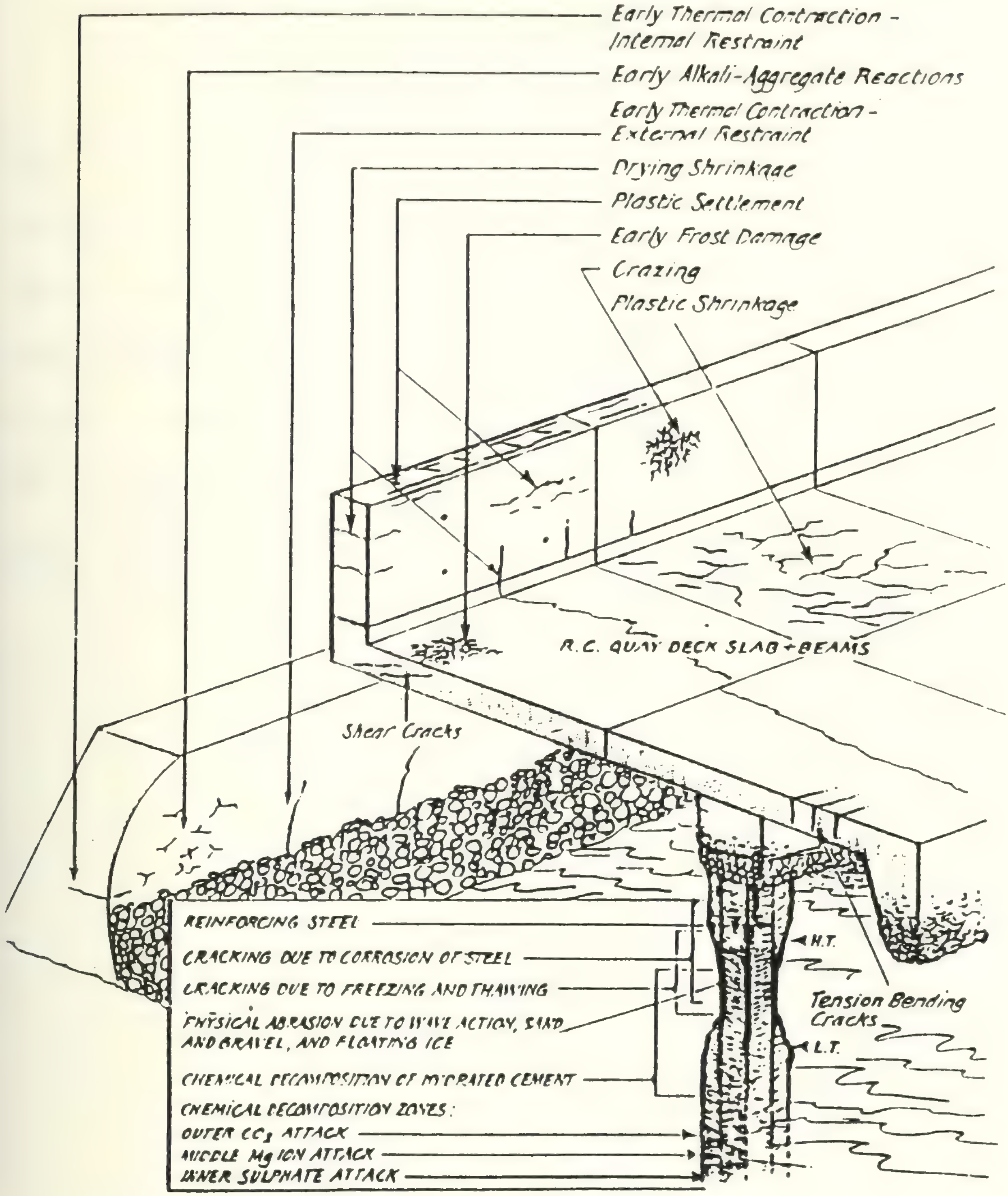


FIGURE IV.2 Summary of Deterioration Modes of Concrete Structures in Seawater

Method	Production Rate (ft^2/min)		Safe Work Duration (hr)
	Preliminary Cleaning	Final Cleaning	
Hand Scraper	0.2	--	2
Cavitation Pistol	1.4	0.6	2
Reactionless Jet	2.3	0.8	2
Sand Injection Jet	1.4	0.4	2
Barnacle Buster	--	0.6	1
NCEL System	3.0	0.7	2
Reaction Jet	3.6	1.1	1

FIGURE IV.3 Diver Production Rates for Underwater Cleaning Using Various Techniques [61]

COMPRESSION LAP SPLICES FOR GRADE 60 BARS—Use 30 bar diameters for all $f'_c \geq 3,000$ psi.

BASIC* TENSION LAP SPLICES FOR GRADE 60 BARS

Bar Size	CLASS A, B & C LAP SPLICE LENGTHS (inches)														
	$f'_c = 3,000$			$f'_c = 3,750$			$f'_c = 4,000$			$f'_c = 5,000$			$f'_c = 6,000$		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
#3	12	12	15	12	12	15	12	12	15	12	12	15	12	12	15
#4	12	16	20	12	16	20	12	16	20	12	16	20	12	16	20
#5	15	20	26	15	20	26	15	20	26	15	20	26	15	20	26
#6	19	25	33	18	24	31	18	24	31	18	24	31	18	24	31
#7	26	34	45	24	31	40	23	30	39	21	27	36	21	27	36
#8	35	45	59	31	40	53	30	39	51	27	35	46	25	32	42
#9	44	57	74	39	51	67	38	49	65	34	44	58	31	40	53
#10	56	72	95	50	65	85	48	63	82	43	56	73	39	51	67
#11	68	89	116	61	80	104	59	77	101	53	69	90	48	63	82

* Normal weight concrete; bars other than top bars.

1. For vertical bars centered in walls, slab bars and temperature bars in slabs or footings with less than 12 in. or concrete below, etc., spaced 6 in. or more, use 0.8 basic lap lengths shown, but not less than 12 in.
2. In standard spiral columns, use 0.75 basic lap lengths shown, but not less than 12 in.
3. For top bars, #4 or larger, multiply lengths above by 1.4.

FIGURE IV.4 Representative Splice Length Data for Reinforcing Steel [45]

	Epoxy Resins Grouts Mortars & Concretes	Polyester Resin Grouts Mortars & Concretes	Cementitious Grouts Mortars & Concretes	Polymer Modified Cementitious Systems
Compressive strength N/mm^2	55 - 110	55 - 110	20 - 70	10 - 80
Compressive modulus E-value, kN/mm^2	0.5 - 20	2 - 10	20 - 30	1 - 30
Flexural strength N/mm^2	25 - 50	25 - 30	2 - 5	6 - 15
Tensile strength N/mm^2	9 - 20	8 - 17	1.5 - 3.5	2 - 8
Elongation at break %	0 - 15	0 - 2	0	0 - 50
Linear coefficient of thermal expansion per °C	25 - 30 $\times 10^{-6}$	25 - 35 $\times 10^{-6}$	7 - 12 $\times 10^{-6}$	8 - 20 $\times 10^{-6}$
Water absorption, 7 days at 25°C, %	0 - 1	0.2 - 0.5	5 - 15	0.1 - 0.5
Maximum service temperature under load °C	40 - 80	50 - 80	In excess 300° dependent upon mix design	100 - 300
Rate of development of strength at 20°C	6 - 48 hours	1 - 2 hours	1 - 4 weeks	1 - 7 days

FIGURE IV.5 General Properties of Four Basic Repair Material Categories [42] (1 of 2)

	Epoxy Resins Grouts Mortars & Concretes	Polyester Resin Grouts Mortars & Concretes	Cementitious Grouts Mortars & Concretes	Polymer Modified Cementitious Systems
Compressive strength N/mm^2	55 - 110	55 - 110	20 - 70	10 - 80
Compressive modulus E-value, kN/mm^2	0.5 - 20	2 - 10	20 - 30	1 - 30
Flexural strength N/mm^2	25 - 50	25 - 30	2 - 5	6 - 15
Tensile strength N/mm^2	9 - 20	8 - 17	1.5 - 3.5	2 - 8
Elongation at break %	0 - 15	0 - 2	0	0 - 50
Linear coefficient of thermal expansion per °C	25 - 30 $\times 10^{-6}$	25 - 35 $\times 10^{-6}$	7 - 12 $\times 10^{-6}$	8 - 20 $\times 10^{-6}$
Water absorption, 7 days at 25°C, %	0 - 1	0.2 - 0.5	5 - 15	0.1 - 0.5

	Epoxy Resins Grouts Mortars & Concretes	Polyester Resin Grouts Mortars & Concretes	Cementitious Grouts Mortars & Concretes	Polymer Modified Cementitious Systems
Maximum service temperature under load °C	40 - 80	50 - 80	In excess 300° dependent upon mix design	100 - 300
Rate of development of strength at 20°C	6 - 48 hours	1 - 2 hours	1 - 4 weeks	1 - 7 days

FIGURE IV. 5

(2 of 2)

Planning and Estimating Data for Concrete Pile Repair Using Concrete Encasement

Description of Task: Repair a deteriorated concrete pile by installing a concrete encasement from 1 foot above the high water line to 2 feet below the mud line. The total length of encasement is 20 to 30 feet. Reinforcement of the pile is not required.

Size of Crew: 2 divers, 2 laborers.

Special Training Requirements: Familiarity with the type of jacket to be used for the concrete form, concrete pump operation, jetting or air lifting procedures, and removal of marine growth.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, hydraulic power unit, concrete pump with adequate hose, concrete mixer (if ready-mix concrete is not available), jetting pump and hose, rigging equipment, float stage, scaffolding.

Productivity of Crew: 6 hours per pile repair.

Materials:

Form Material

Either flexible or rigid forms may be used. When using proprietary forms, follow manufacturers' recommendations regarding lengths and diameter of forms, top and bottom closures, spacers, bands, straps, and special fittings. Forms are ordered prefabricated in the required length and diameter. For flexible forms, allowance on the length must be made for extra fabric that may be required around blocking at the top and bottom of the jacket. Some proprietary systems require that different types of forms be used in the tidal and submerged zones.

Spacers

A conservative estimate of the number of spacers must be made. In calm water and with vertical piles, relatively few spacers will be required. Rough water and batter piles will require more spacers.

Wire Mesh Reinforcing

Usually 6x6-10/10 welded wire fabric is adequate. Calculate the width of wire fabric required based on its circumference, taking into consideration the thickness of the spacers between the pile and reinforcing and allowing a 9-inch overlap of the ends.

FIGURE IV.6 Planning and Estimating Data For Concrete Pile Repair - Concrete Encasement [26] (1 of 2)

Concrete

To determine the amount of concrete required to fill the form, be conservative. When using flexible jackets allow for reduction of concrete volume due to loss of water through the permeable fabric, enlargement of the jacket caused by stretching, and waste. Usually an allowance of 10% extra concrete over the theoretically calculated quantity is sufficient.

Form Reinforcing Straps and Special Fittings

Rigid forms usually require reinforcing straps. The spacing and number required depend on the type of form and the hydrostatic pressure of the concrete fill. Some types of reinforcing straps are reusable, but an allowance should be made for a loss of between 10 and 20% of the straps each time they are used. In addition to reinforcing straps, closure forms, blocking hangers, inlet valves, and clamps will be required, the number and type depending on the forming system being used.

Potential Problems: Potential ripping of fabric forms requires familiarity with repair procedures and the availability of repair equipment. Potential unzipping or unlocking of form seams requires familiarity with banding or strapping procedures with form partially full of concrete.

FIGURE IV.6

(2 of 2)

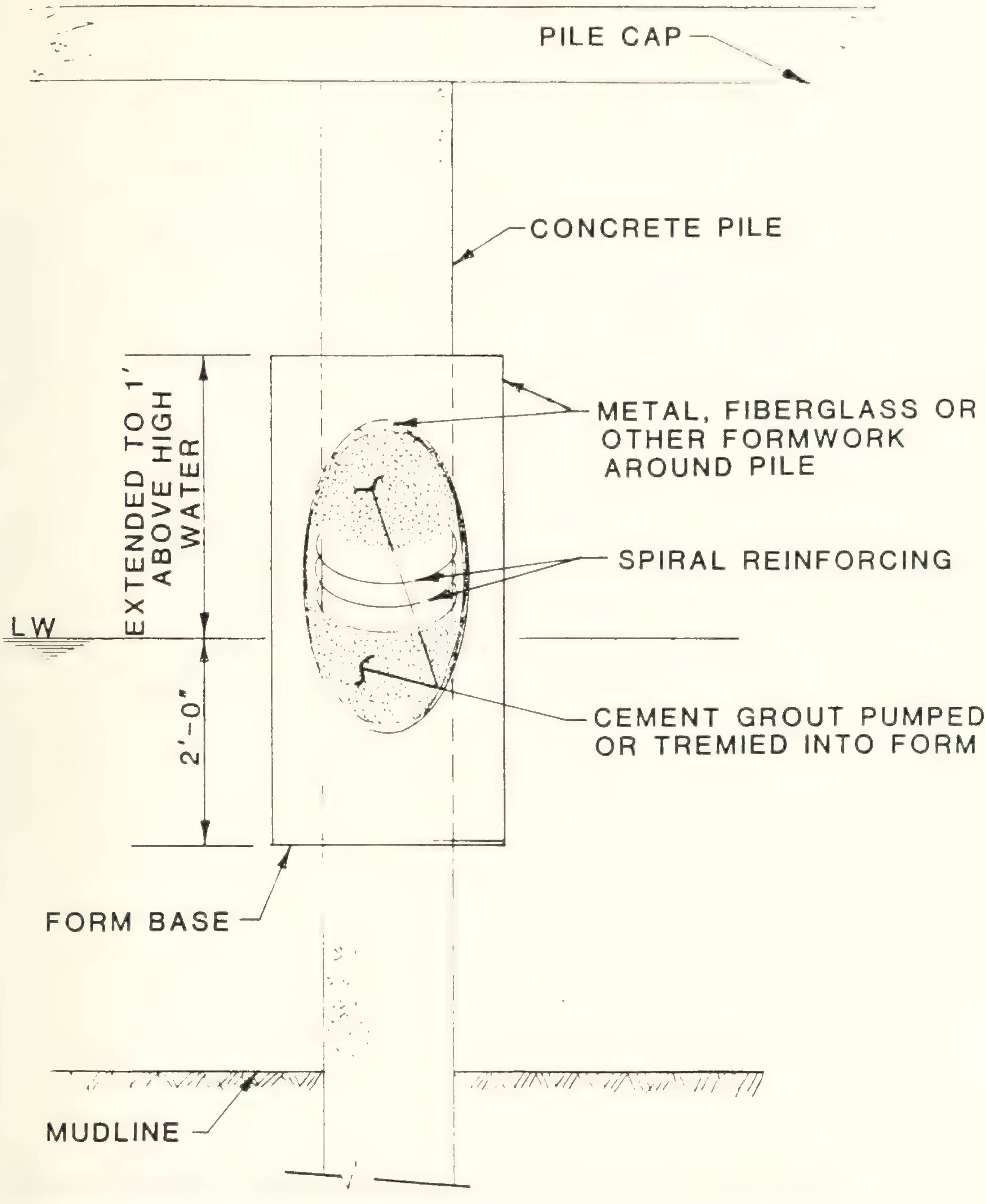


FIGURE IV.7 Typical View of Concrete Encasement Method [26]

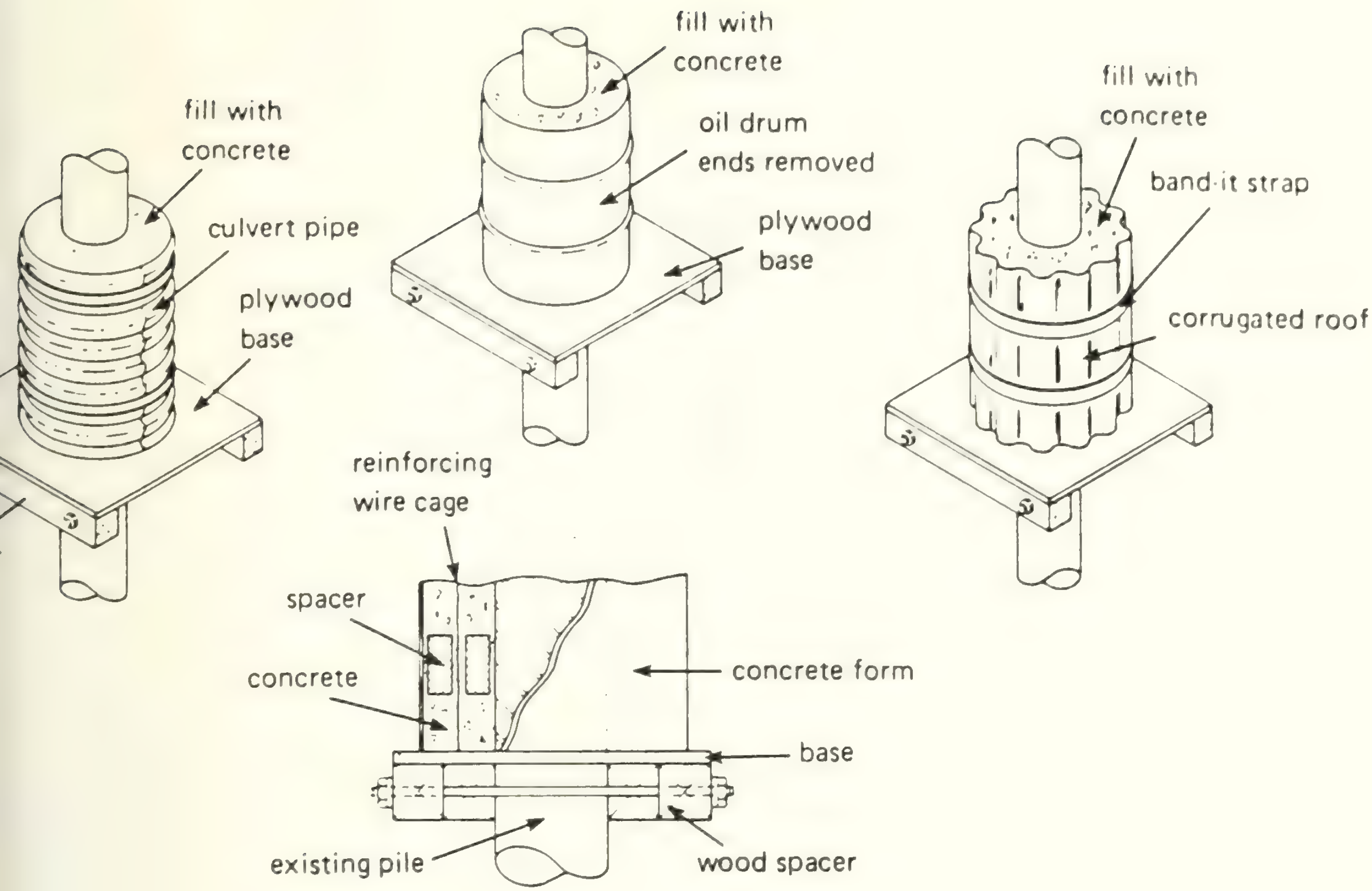


FIGURE IV.8 Alternative Forming Methods for Concrete Encasement

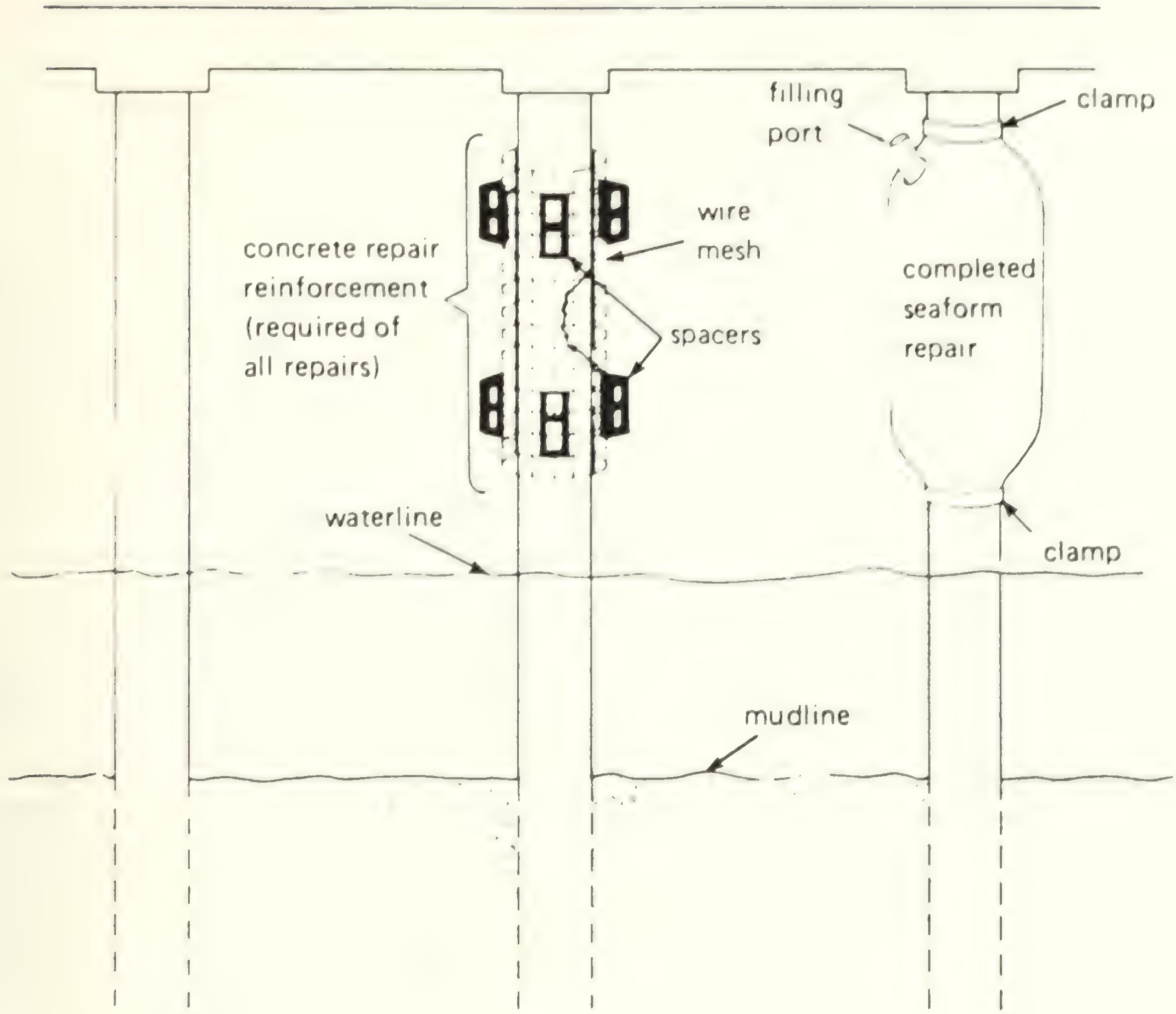


FIGURE IV.9 "Seaform" Repair Technique for Concrete Encasement

CONCRETE PILES

Repairs	Repair abrasion of concrete piles with concrete jacketing. Clean concrete pile to remove growth and deteriorated concrete. Wrap pile with spiral reinforcing in area where concrete jacket is to be cast. Place form around pile (metal, fiber, fiberglass or other material). Pump or tremie 5000 psi concrete inside form. Remove form after concrete has hardened or leave in place for extra protection.
Claims	Protects concrete pile from further damage by abrasion.
Problems	Quality control essential to get durable concrete.
Environment	Fresh or salt water
Special tools	Concrete pump
Divers	Required
Note:	Can be used to repair timber and steel piles.

FIGURE IV.10 Generic Repair Procedure For Concrete Encasement

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION	ITEM NO. 838	Page 1 of 2
BRIDGE REPAIR MANUAL	UNIT: Each	

ITEM NAME: Concrete Pile Jacket (Reinforced)

DESCRIPTION: Encasement of concrete pile with a concrete jacket reinforced with epoxy coated rebars.

APPLICATION: This repair method should be used where a concrete pile has deteriorated to the point that structural integrity of the pile is in question.

TRAFFIC CONTROL AND SAFETY: Refer to Manual on Uniform Traffic Control & Safe Practices.

MATERIALS: Class III Concrete -------------- M.S. #354-3
Concrete Spacers
Reinforcing Steel (Epoxy Coated)- M.S. #931-1

CONSTRUCTION METHOD:

1. Remove all cracked and unsound concrete.
2. Clean pile surfaces of oil, grease, dirt and other foreign materials which would prevent proper bonding.
3. Sandblast exposed reinforcing steel to "near-white metal".
4. Place reinforcing steel cage around pile.
5. Set forms for concrete jacket. (Treat forms with an approved form release agent before placing concrete.)
6. Dewater forms and place concrete.
7. Leave forms in place for a minimum of 72 hours.

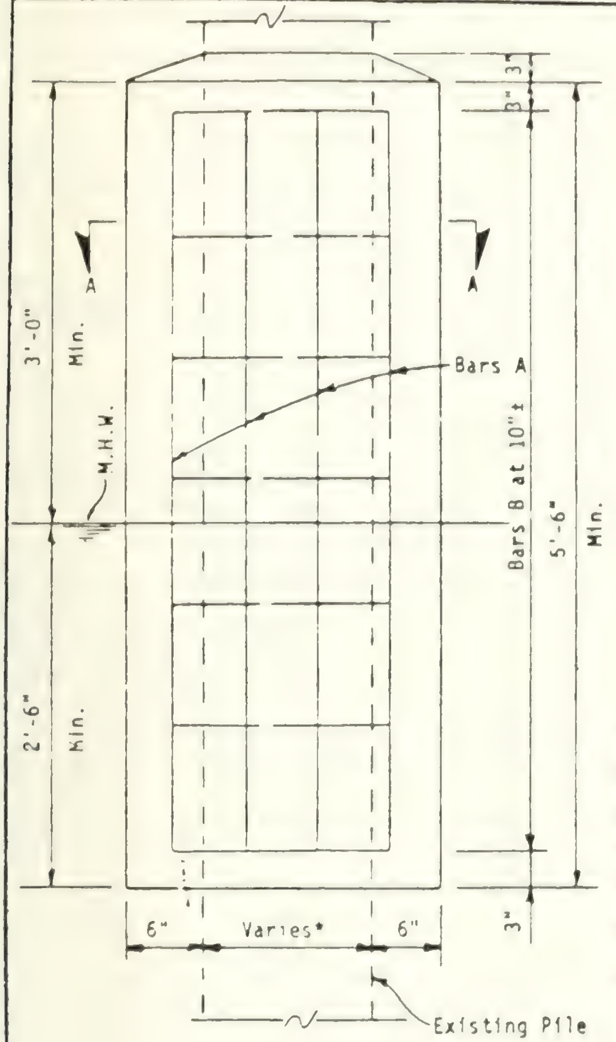
Typical condition where this repair method should be used.

FIGURE IV.11 Concrete Encasement Procedure [47]

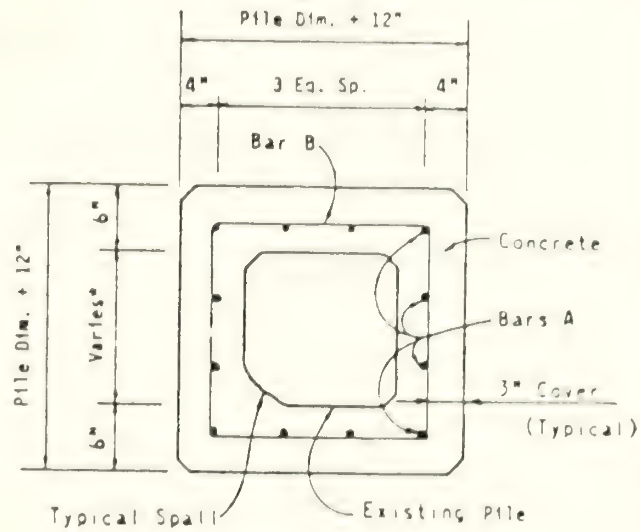
(1 of 2)

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE REPAIR MANUAL	ITEM NO. 838	Page 2 of 2
	UNIT: Each	

ITEM NAME: Concrete Pile Jacket (Reinforced)



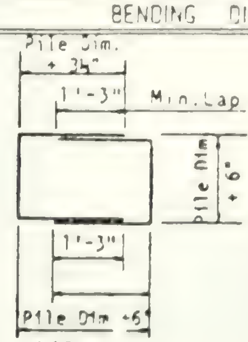
ELEVATION OF PILE



SECTION A-A

BILL OF REINFORCING STEEL			
MARK	SIZE	NO. REQ'D.	LENGTH
A	5	12	Min. 5'-[illegible]
B	4	Min. 7	Varies

BENDING DIAGRAMS



BAR B

NOTE: All bar dimensions are out-to-out.

FIGURE IV.11

(2 OF 2)

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION	ITEM NO. 839	Page 1 of 5
BRIDGE REPAIR MANUAL	UNIT: Linear Foot	

ITEM NAME: Integral Pile Jacket (Conc.) - Type I, II, III, IV, V & VI

DESCRIPTION: Encasement of concrete piles with a fiberglass form filled with epoxy grout, cement grout or seal concrete between form and pile.

APPLICATION: Types I through IV are used to protect the pile from further deterioration. Types V and VI are used to restore structural integrity and protect the pile from further deterioration. See note on Page 2.

TRAFFIC CONTROL AND SAFETY: Refer to Manual on Uniform Traffic Control & Safe Practices.

MATERIALS:

Material	Type	Spec
Epoxy bonding compound		APL
Fiberglass Forms	All types	M.S. #457-1
Epoxy Grout Filler	Type I & III	M.S. #926-2
Portland Cement Grout Filler	Type II & IV	M.S. #460-30
Class III Concrete Filler	Type V & VI	M.S. #345-3

CONSTRUCTION METHOD:

General for all Types:

1. Clean pile surfaces of oil, grease, dirt and other foreign materials which would prevent proper bonding.
2. Remove cracked and unsound concrete.
3. Sandblast exposed reinforcing steel to "near-white metal".
4. Place pile jacket form around pile. Standoffs of either form material or concrete blocks should be permanently attached to form or pile.
5. Seal interlocking joint with epoxy bonding compound and seal bottom of form against pile surface.
6. Place external bracing and bonding materials.
7. Dewater form.

Typical condition where Types I and III jacket should be used.

FIGURE IV.12 Encasement of Concrete Pile Using Fiberglass Form

8. Fill annulus between the pile and form with specified filler.

9. Remove external bracing and banding, and clean any filler material deposited on forms.

SPECIAL CONSTRUCTION METHODS

Prestressed Concrete Pile - In cases where prestressed strands have been exposed and deteriorated, they are to be removed and replaced with No. 4 reinforcing bars. A minimum of 12" of sound strand material should be used at each end of reinforcing bar for splice.

NOTE:

Type I, III and V are used when deterioration terminates at the water line or no more than one or two feet below.

Type II, IV and VI are used when deterioration runs the full length of the pile.

Typical condition where Types II and IV jacket should be used.

Typical condition where Types V or VI jacket should be used.

FIGURE IV.12

(2 of 5)

ITEM NAME: Integral Pile Jacket (Conc.) - Type I, II, III, IV, V & VI

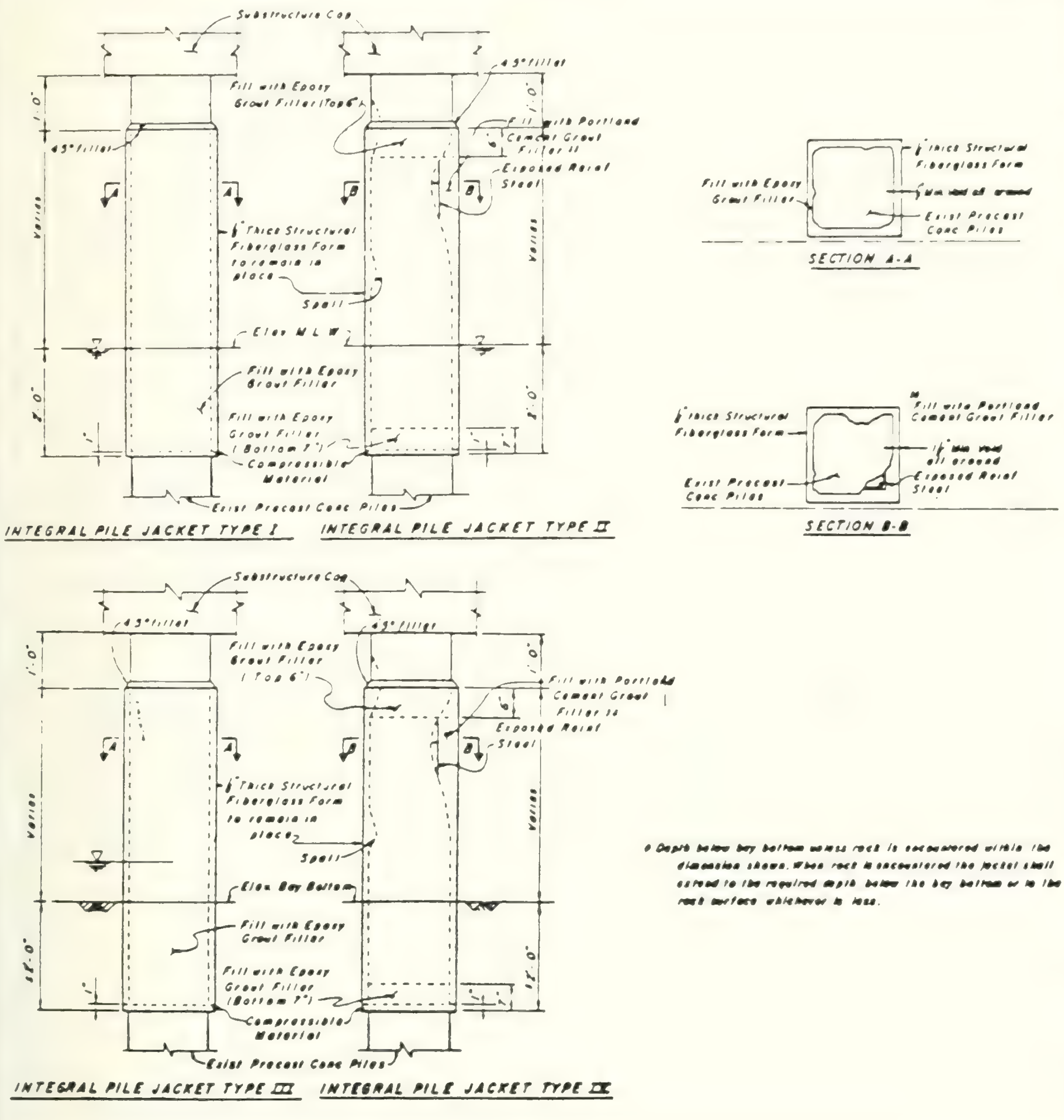
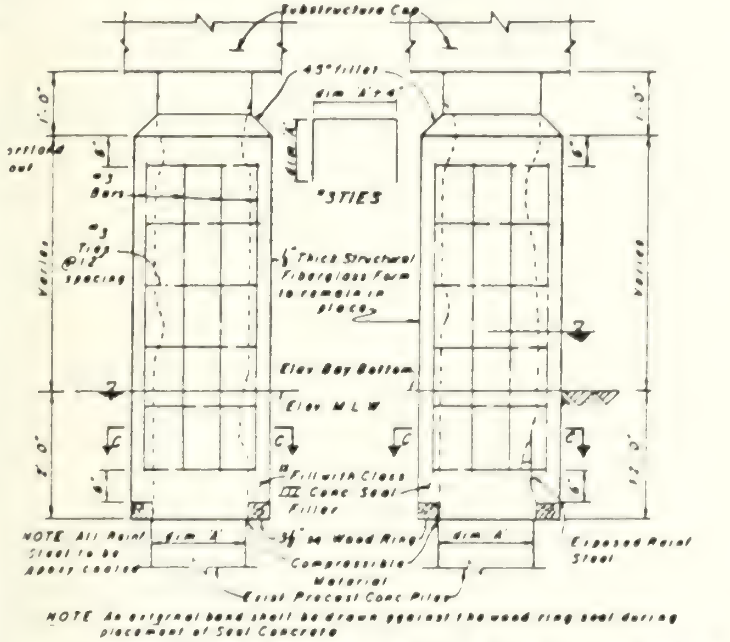


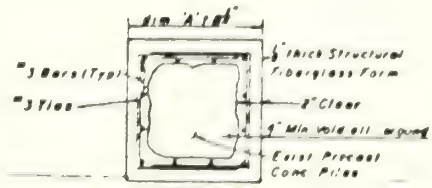
FIGURE IV.12

(3 of 5)

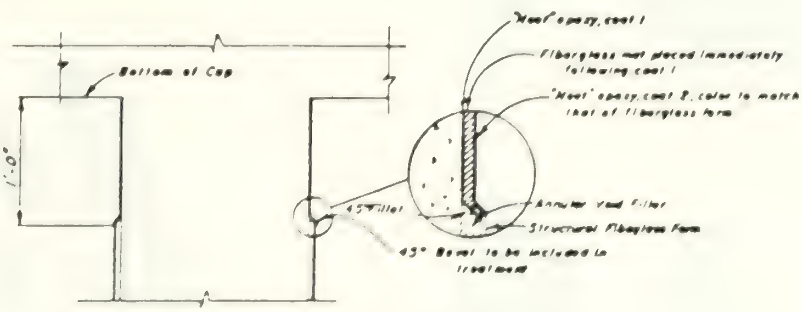


NOTE An original bond shall be drawn against the wood ring seal during placement of Seal Concrete

INTEGRAL PILE JACKET TYPE V   INTEGRAL PILE JACKET TYPE VI



SECTION C-C



DETAIL OF STANDARD TREATMENT OF TOP 12" OF PILING

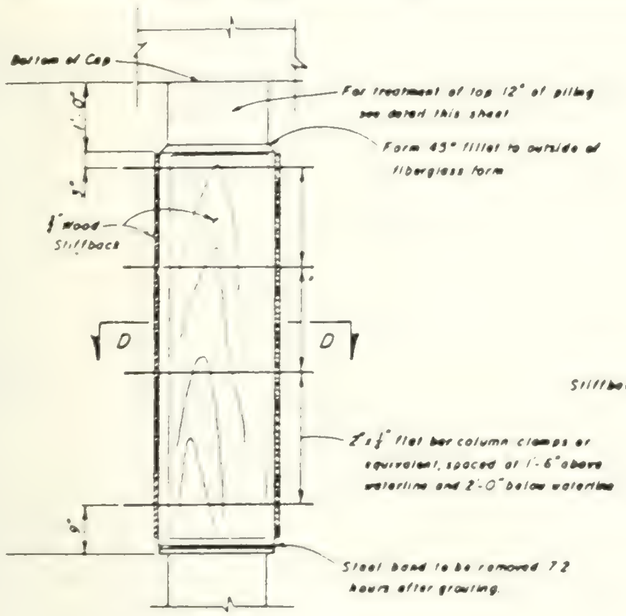
NOTE: The top 12" of piling shall be restored to the original lines with epoxy mortar.

FIGURE IV.12

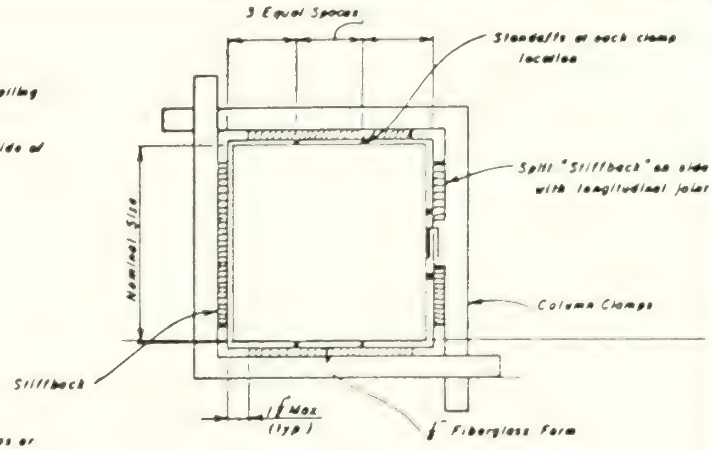
(4 of 5)

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION	ITEM NO. 839	Page 5 of 5
BRIDGE REPAIR MANUAL	UNIT: Linear Foot	

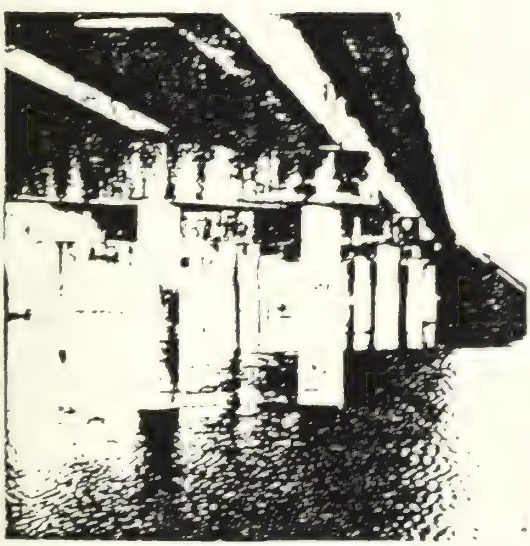
ITEM NAME: Integral Pile Jacket (Conc.) - Type I, II, III, IV, V & VI



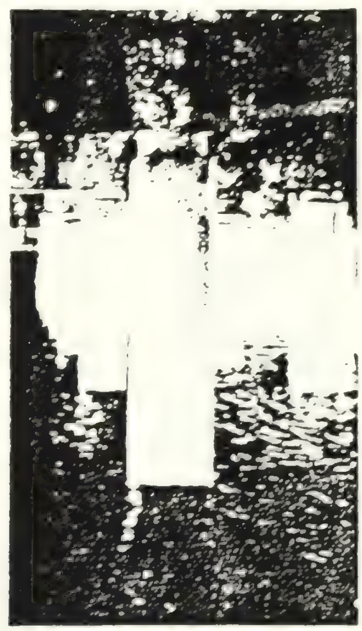
CONSTRUCTION METHOD DETAIL



SECTION D-D



Example of Integral Pile Jacket constructed on a round pile.



Example of Integral Pile Jacket constructed on a square pile.

FIGURE IV.12

(5 of 5)

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE REPAIR MANUAL	ITEM NO. 840	Page 1 of 3
	UNIT: Linear Foot	

ITEM NAME: Integral Pile Jacket - Steel

DESCRIPTION: Encasement of steel piles with a fiberglass form filled with Portland cement grout filler between form and pile.

APPLICATION: Provides protection to steel piles above and below water. Not applicable where loss of section of the pile is such that reinforcement is required.

TRAFFIC CONTROL AND SAFETY: Refer to Manual on Uniform Traffic Control & Safe Practices.

MATERIALS: Fiberglass Forms ------------- M.S. #457-1
Epoxy Grout Filler ----------- M.S. #926-2
Portland Cement Grout Filler - M.S. #460-30
Epoxy Bonding Compound ------- APL

CONSTRUCTION METHOD:

1. Clean pile surfaces of oil, grease, dirt and corrosion by sandblasting to "near white metal".
2. Place pile jacket form around pile. Standoffs of form material should be permanently attached to form or pile.
3. Seal interlocking joint with epoxy bonding compound and seal bottom of form against pile surfaces.
4. Place external bracing and banding materials.
5. Dewater form.
6. Fill bottom six inches of form with epoxy grout filler.
7. Fill form to top 6" with Portland cement grout filler.
8. Fill to 6" with epoxy grout filler. Form fillet and slope grout to web of pile.
9. Remove external bracing and banding, and clean any filler material deposited on form.

Typical condition where this repair method should be used.

FIGURE IV.13 Encasement of Steel Piles With Underwater Concrete (Grout) Placement [46] (1 of 3)

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

BRIDGE REPAIR MANUAL

ITEM NO. 840

UNIT: Linear Foot

ITEM NAME: Integral Pile Jacket - Type VII & VIII

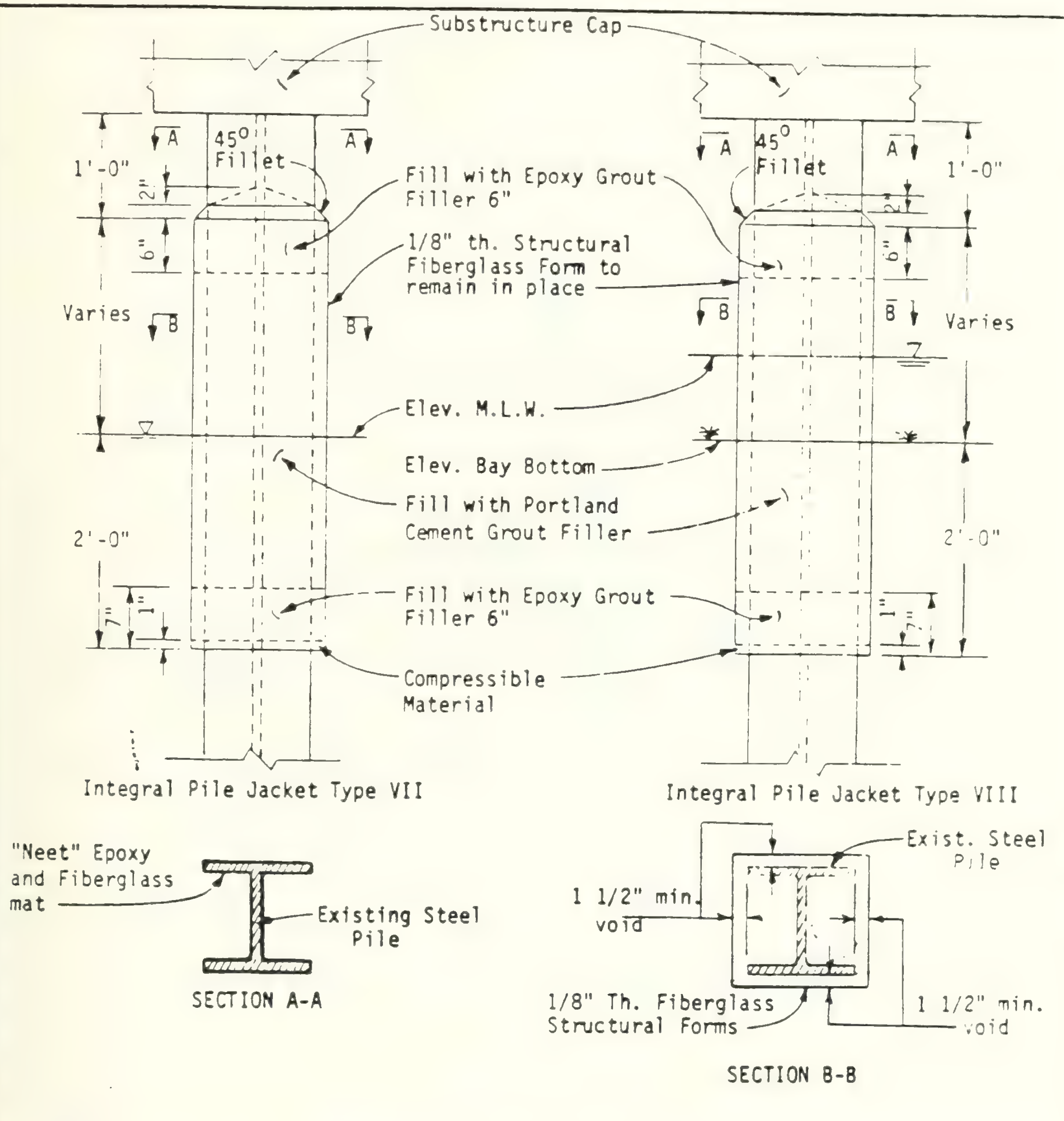
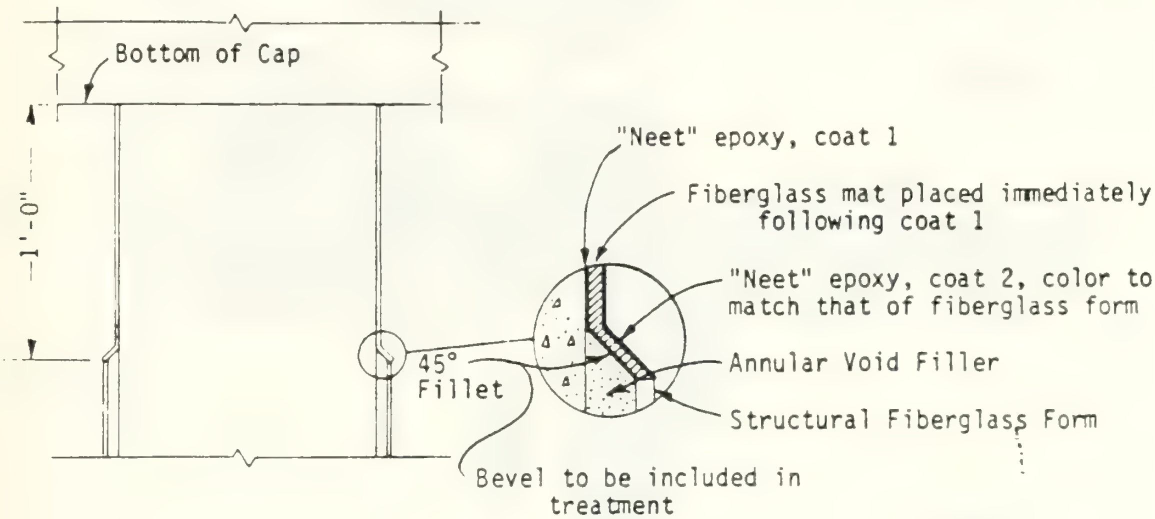


FIGURE IV.13

(2 of 3)

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION	ITEM NO. 840	Page 3 of 3
BRIDGE REPAIR MANUAL	UNIT: Linear Foot	

ITEM NAME: Integral Pile Jacket - Steel



DETAIL OF STANDARD TREATMENT OF TOP 12" OF PILING

NOTE: The top 12" of piling shall be restored to the original lines with epoxy mortar.

FIGURE IV.13

(3 of 3)

FIGURE IV.14 Epoxy Grouting Using Hand Caulking Gun [54]

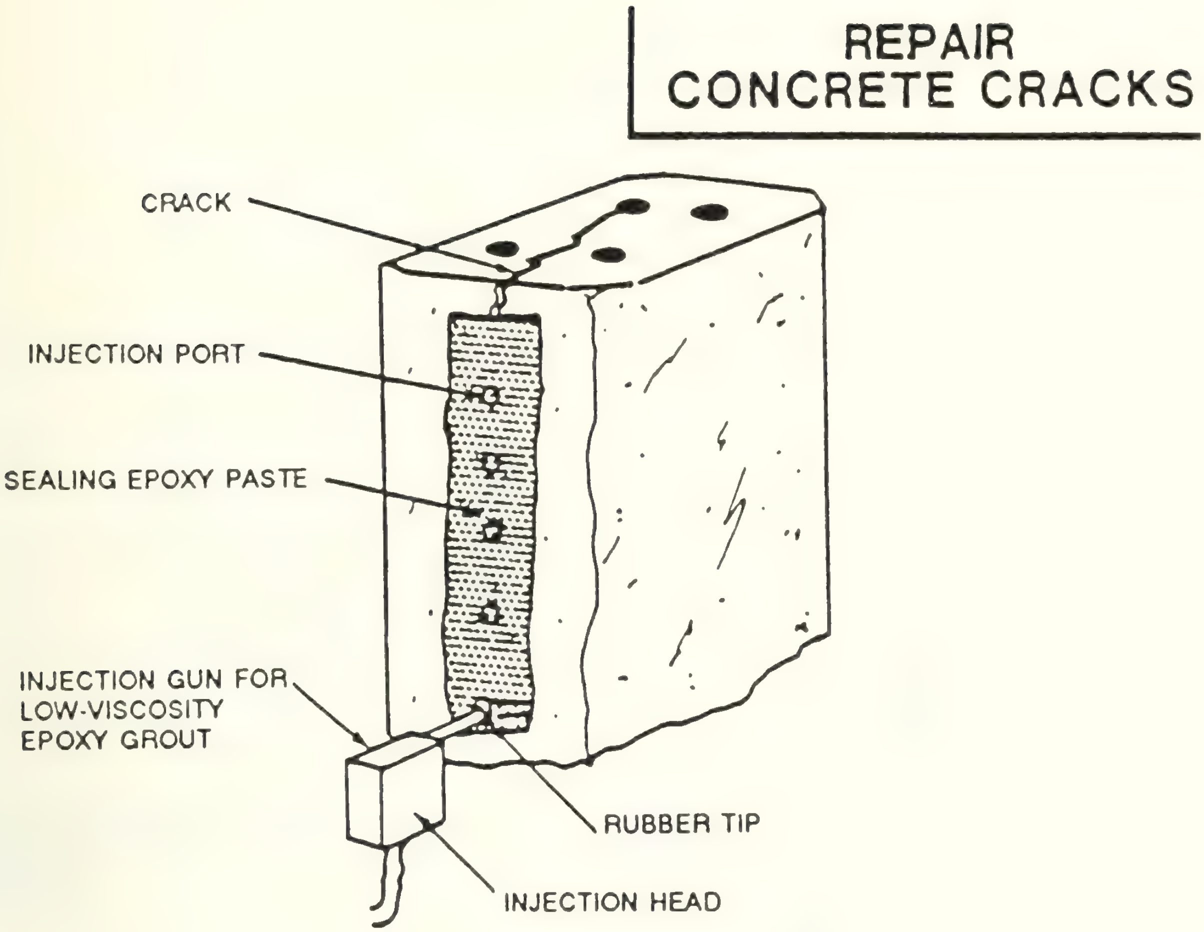


FIGURE IV.15 Typical Arrangement for Concrete Crack Repair Using Epoxy Injection [54]

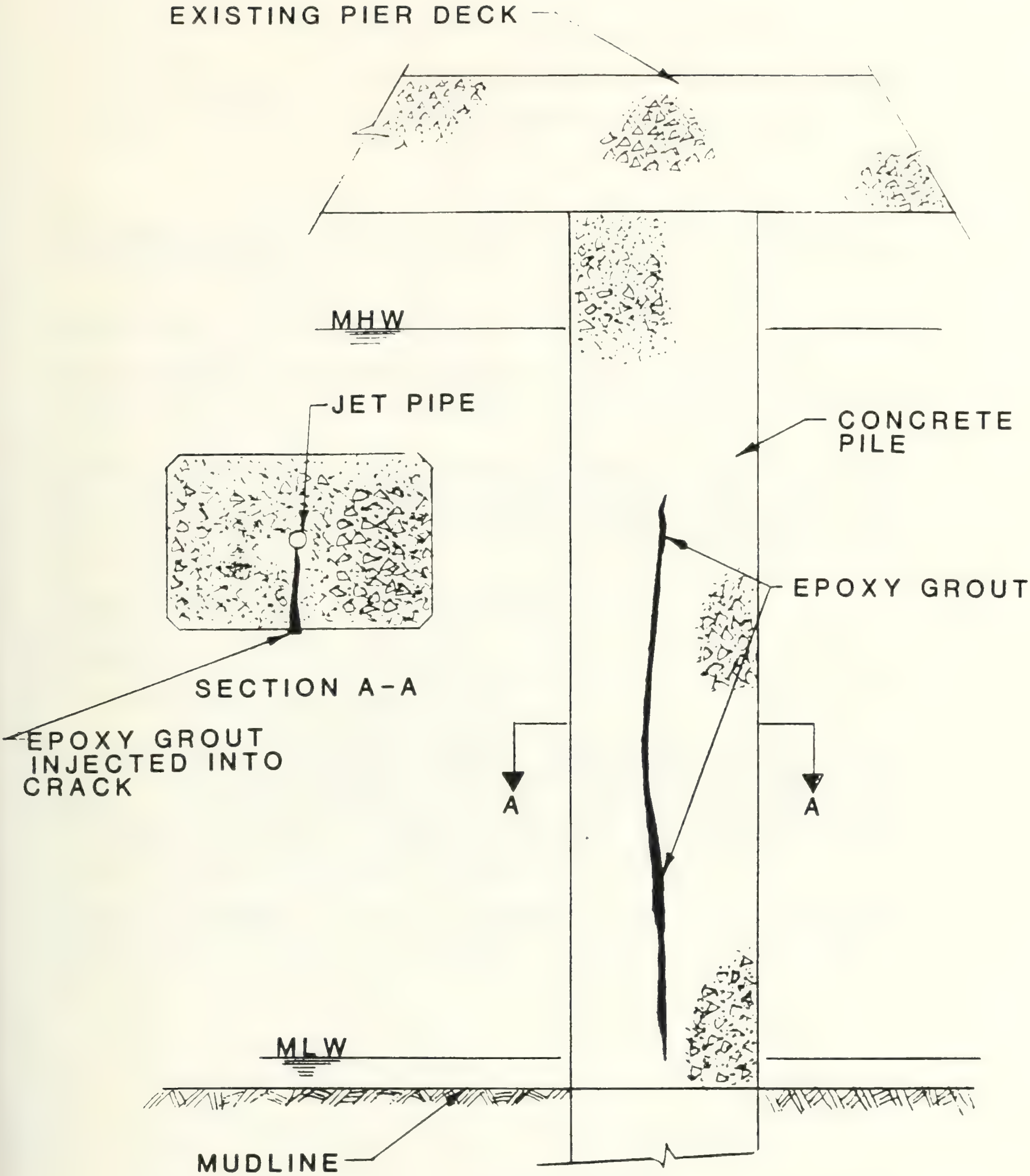


FIGURE IV.16 Epoxy Injection Arrangement For Damaged Concrete Pile [26]

Planning and Estimating Data for Concrete Pile Repair Using Epoxy Injection

Description of Task: Repair a 6-inch-deep crack in a concrete pile by injecting low-viscosity neat epoxy grout into the crack. Total length of crack to be repaired is 10 feet.

Size of Crew: 2 divers, 1 laborer.

Special Training Requirements: Familiarity with procedures for removal of marine growth, the use of epoxy grout pump, and the use of injectable epoxy.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, hydraulic drill with bits, high-pressure pump for waterblaster, hydraulic power unit, protective clothing for personnel handling the epoxy compound, epoxy pump, float stage or work platform.

Productivity of Crew: 10 min/linear ft of crack.

Materials:

Low-Viscosity Epoxy Grout

Commercially available injectable epoxy grouts are usually purchased in two-component kits. Mixing proportions will vary, so manufacturers' instructions should be followed. The volume of the crack must be estimated by taking its length, average width, and average depth. An additional 25% should be added to allow for overfilling of the cracks and inaccuracies in estimating the size of the cracks.

Potential Problems: If water temperature is less than 60°F, proper adhesion to the pile may not occur. Skin irritation may occur if individual is sensitive to the epoxy material.

FIGURE IV.17 Planning And Estimating for Concrete Pile Repair Using Epoxy Injection [26]

CONCRETE PILES

Repairs	Filling cracks in concrete piles with epoxy adhesive. Wire brush around cracks to be pumped. Use high water pressure to clean cracks. Place nails in cracks for use as ports while pumping epoxy. Apply a minimum amount of sealer to surface of crack and allow for curing time. Use a high modulus, low viscosity rigid epoxy adhesive insensitive to moisture. Using special pump that mixes epoxy components just prior to injection; inject grout under pressure until crack is full and/or back pressure increases to pump pressure. The plugging of port holes after injection is important to prevent leakage.
Claims	Seals cracks in concrete piles above or below water.
Problems	Epoxy will not set in low temperature (below 40°F). Protective clothing must be worn by workers handling epoxies.
Environment	Fresh or salt water
Special tools	Pump which mixes epoxy just before injection
Divers	Required

FIGURE IV.18 Epoxy Injection Repair Procedure [26]

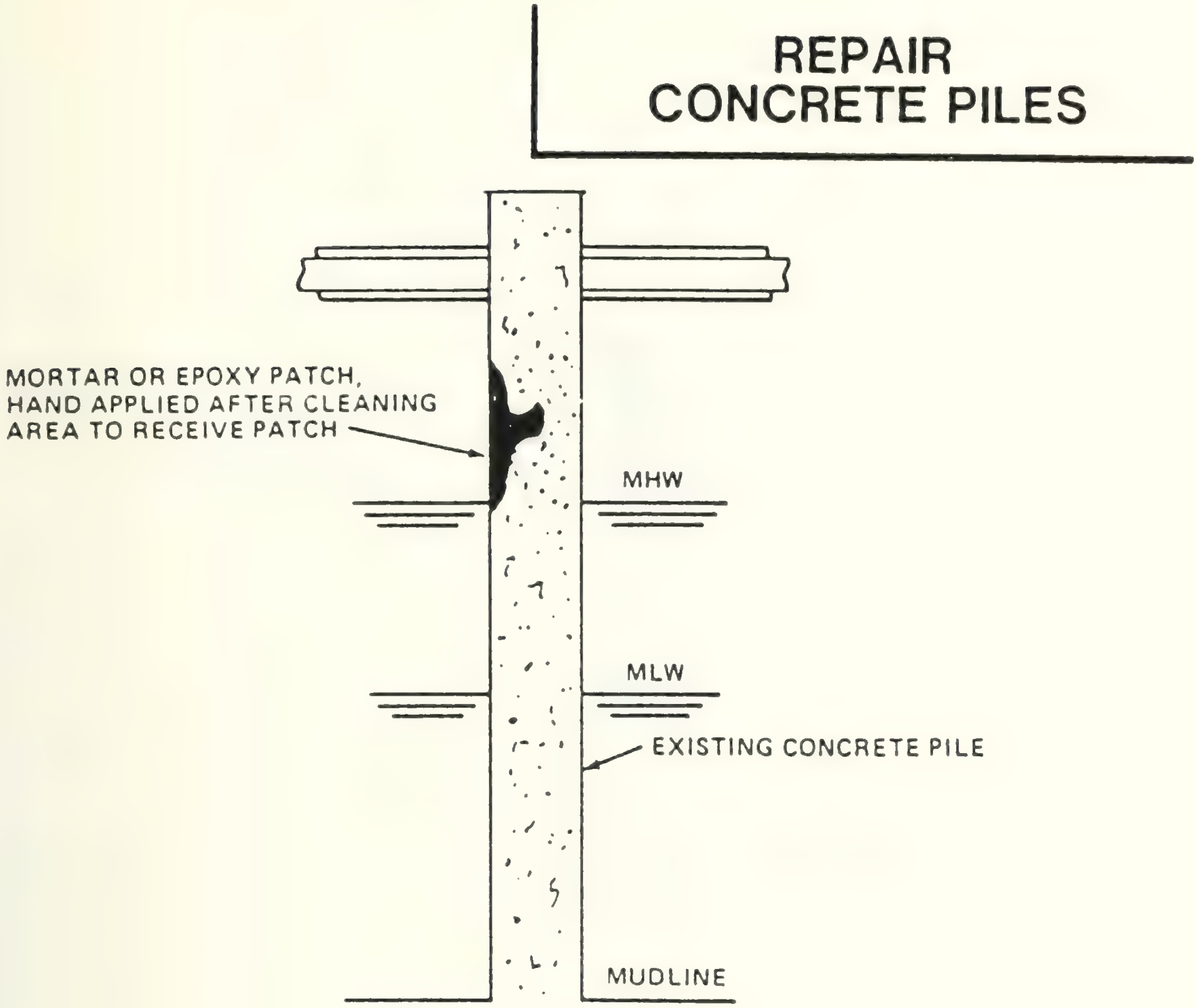


FIGURE IV.19 Patching Method for Repairing Concrete Piles [61]

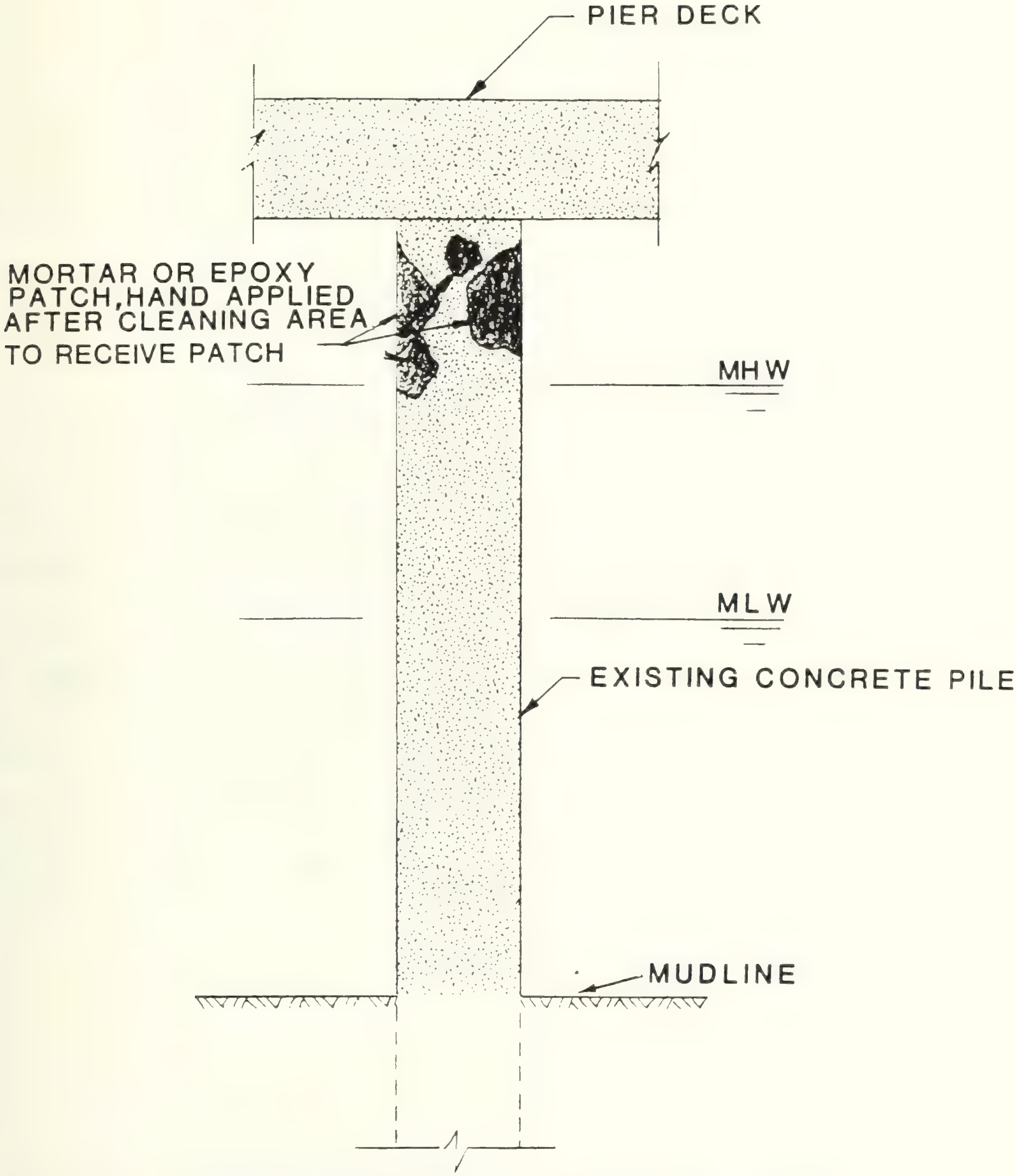


FIGURE IV.20 Hand Applied Patch Material on Concrete Pier Pile

CONCRETE PILES

Repairs	Mortar patching of spalled concrete. Clean area to be patched to sound concrete. Mix mortar using: 1) Cement (fast setting) 95% } 1 part Volcanic clay 5% Sand 2 parts 2) Epoxy with cement and sand fillers. Spread over areas to be patched by smearing with the hand.
Claims	Fills in areas of concrete where spalling has occurred to protect reinforcing.
Problems	Spalling may continue since patching is not any more durable than its ability to adhere to the old concrete.
Environment	Salt or fresh water
Special tools	Protective clothing needed for workers handling epoxy
Divers	Not required
Note:	Can be used to repair concrete walls.

FIGURE IV.21 Procedure for Mortar Patching of Spalled Concrete

Planning and Estimating Data for Concrete Pile Repair Using Epoxy Patching

Description of Task: Repair a deteriorated concrete pile by patching with hand-applied epoxy. Unit area to be repaired is 1 ft^2 underwater.

Size of Crew: 2 divers, 1 laborer.

Special Training Requirements: Familiarity with procedures for removal of marine growth and application of epoxy patching compounds underwater.

Equipment Requirements: High-pressure waterblaster, hydraulic grinder with Barnacle Buster attachment, high-pressure pump for waterblaster, hydraulic power unit, protective clothing for personnel handling the epoxy patching compound, float stage or work platform.

Productivity of Crew: 15 min/ft^2 underwater.

Materials:

Epoxy Patching Compound

Epoxy patching compounds are usually purchased in two-component kits, with an aggregate additive. A 1-gallon kit might include 1 gallon of each component plus aggregate, resulting in more than a 2-gallon yield. Patching coverage is measured in square feet per gallon. The required patching yield is obtained by taking the square footage to be covered and dividing by the square foot per gallon coverage rate.

Potential Problems: If water temperature is less than 60°F, proper adhesion to the pile may not occur. Skin irritation may occur if individual is sensitive to the epoxy material.

FIGURE IV.22 Planning and Estimating for Concrete Pile Repair Using Epoxy Patching [26]

CONCRETE STRUCTURES

Repairs	Covering damaged concrete surfaces by pneumatically projected concrete. Clean concrete surface by chipping. Repair reinforcing by welding new bars to remaining sound reinforcing. Add welded wire fabric over all surfaces to be covered. Anchor fabric at the rate of three anchors per two square feet. Use 1:3 portland cement/sand mix with curing, quick setting and bonding additives. Water/cement ratio from 0.35 to 0.50. Minimum compressive strength of 3000 p.s.i. Before application, wet all bonding surfaces with fresh water. Apply concrete mixed dry and pumped through a hose. Water is added at hose nozzle during application.
Claims	Protects reinforcing by replacing cover. Seals cracks.
Problems	Requires knowledgeable contractor and rigid quality control.
Environment	Salt or fresh water
Special tools	Pneumatically projected concrete pump, hoses and nozzles
Divers	Not required

FIGURE IV.23 Procedure for Shotcrete Repair of Concrete Structures [26]

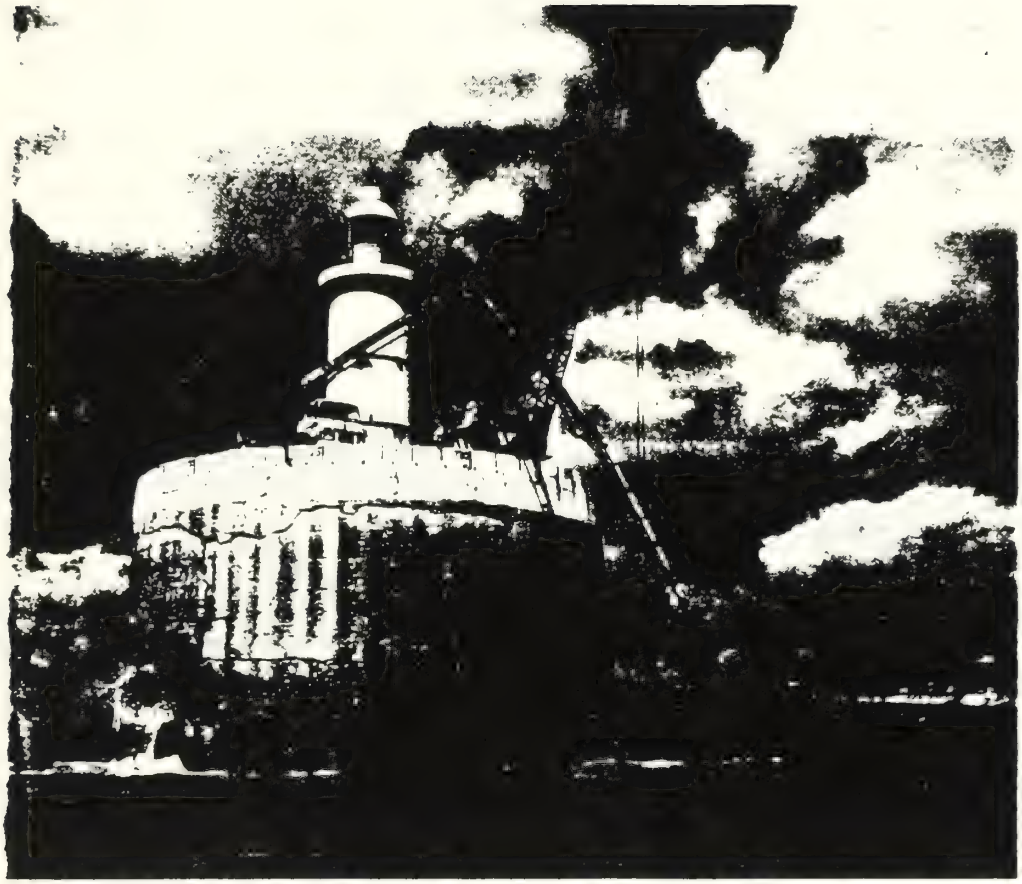


FIGURE IV.25 Pumped Placement of Underwater Concrete at a Harbor Entrance [55]

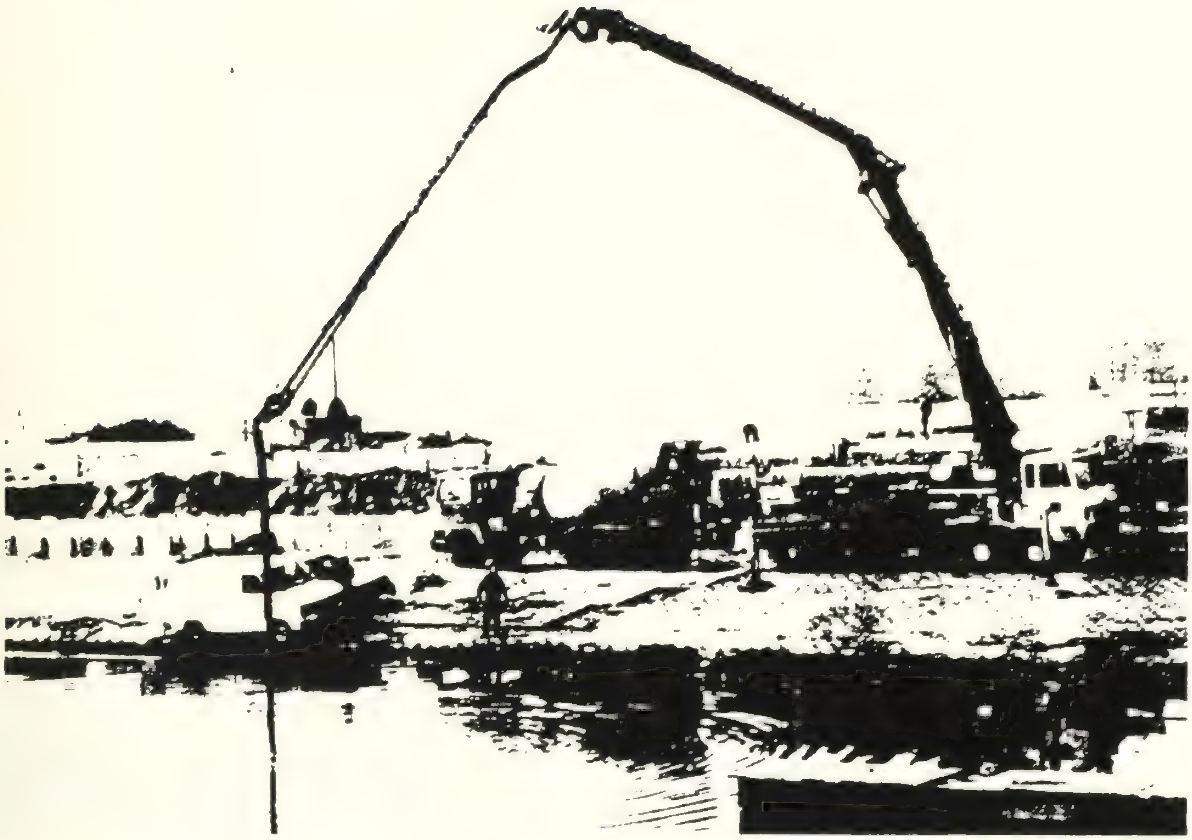


FIGURE IV.26 Example of Using Pumped Tremie Concrete to Access an Underwater Repair [55]

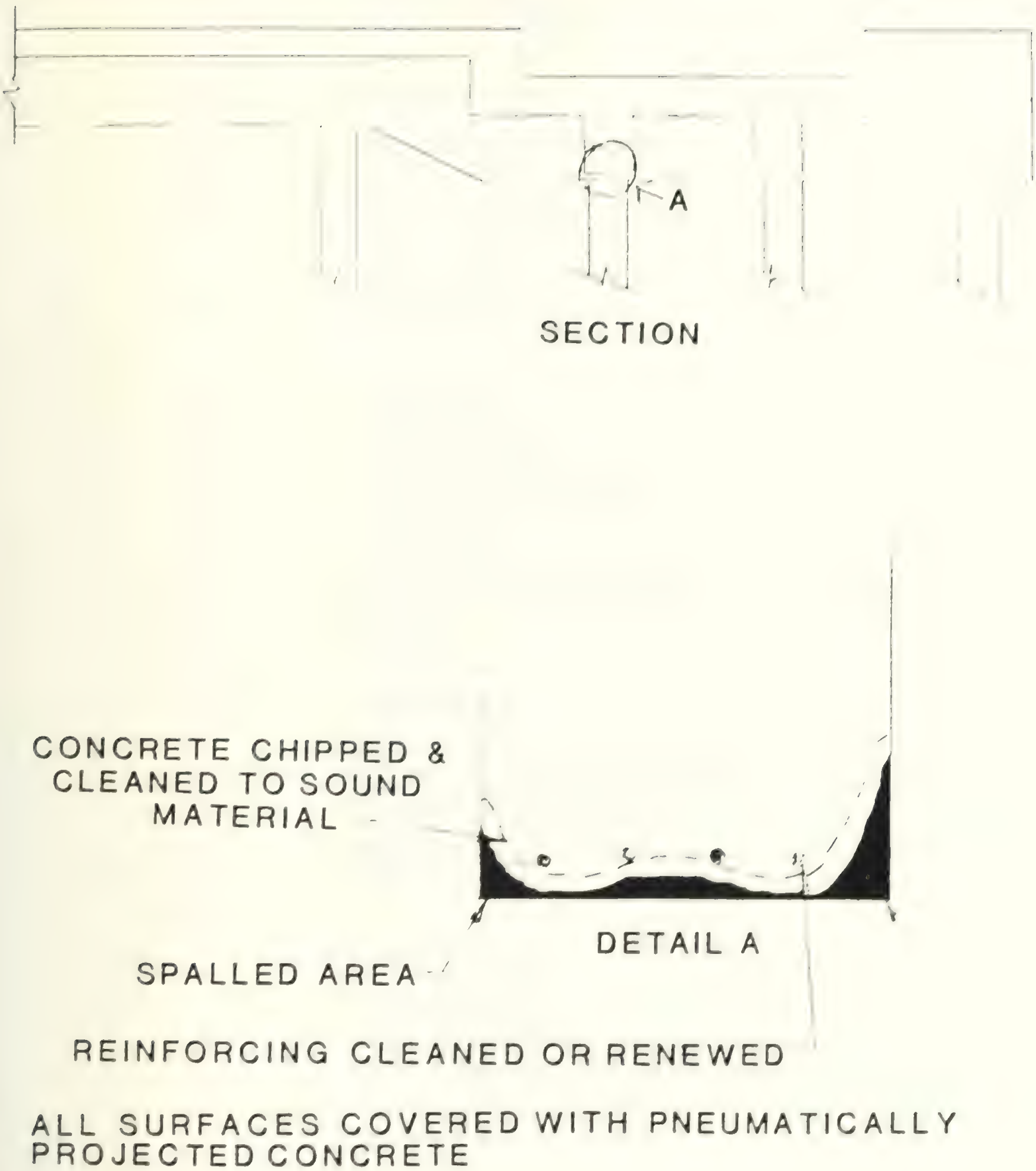


FIGURE IV.24 Schematic View of Shotcrete Repair to a Pier Substructure [41]

10.0 Hints for the site

Starting distance

For vertical placing there must be a starting distance between the concrete pump and the vertical pipeline. It should at least be 10% to 15% of the max. vertical placing distance.

Pumping start

Before starting to pump concrete it should be ensured that the concrete pump and any placing boom used are in good working order and that the pipeline is free. In case of long delivery lines or heated pipes water should be pushed through the pipeline prior to pumping. Take care of the quantities of lubricating grout.

Blowing-out

Blowing-out should be performed only with the catch basket attached to the end of the pipeline – danger of accidents. For this purpose any bends at the end or the end hose must be removed.
A paper plug or jute sack should be used in front of the hard sponge rubber ball. Leaky pipelines impede the blowing-out process.

Pushing-out

It is easier to push long horizontal pipelines and vertical lines out. If the plug is not tight there is a danger of blockage. It is made of soaked empty cement bags and hard sponge rubber balls and should be prepared in a 1 m long pipe section before concreting starts.

Catch basket

When blowing-out the pipeline with air a catch basket must be used at the end of the pipeline. It acts as a shut-off valve. The hard rubber sponge ball held by the catch basket prevents a rapid escape of compressed air. The air must always escape at the pipe cleaning head.

FIGURE IV.27 Hints for Placement of Pumped Concrete [52]

(1 of 7)

Flow concrete

Generally flow concrete is pumpable whenever it is pumpable without concrete additives (super-liquifier).

Pumping height

The pumping height depends on the max. placing pressure of the concrete pump. Use appropriately dimensioned pipelines and couplings. Pumping heights of 500 m are possible.

Horizontal placing distance

A horizontal placing distance of more than 1000 m has already been achieved by a SCHWING concrete pump. The max. possible horizontal distance depends among other factors on the placing pressure of the concrete pump, on the diameter of the pipeline, etc. In case of great horizontal distances take also the quantity of concrete contained in the pipeline into account. Make the necessary preparations for cleaning the pipeline.

Descending pipeline

With descending pipelines it must be ensured that the concrete does not break and segregate in the descending line. The output of the concrete pump must always be higher than what the pipeline can swallow normally. A shut-off valve should be installed after the descending line which prevents the pipeline from running empty if pumping is interrupted.

Compressor

For blowing-out the pipeline compressors with a suction volume of more than 0.8 m^3/min. are suitable. Where the distance compressor/pipeline is more than 20 m, the connection to the pipe cleaning head should be made via two compressed air hoses. Pressure drop!!

Consistency

Pumpable concrete has a consistency in the range of K2 or K3. Avoid changes in concrete consistencies. Concrete in the consistency range K1 can only be pumped if its consistency can be determined by the spread measure.

FIGURE IV.27

(2 of 7)

Couplings

Cup-tension couplings transfer high pressure safely. Stirrup couplings are faster to assemble, they are therefore used if the pipeline has to be repositioned quite frequently.

Light-weight concrete

Light-weight concrete with a bulk density of less than 2 t/m^3 is only pumpable to a certain extent. Aggregates for light-weight concrete must be pretreated – watered.

Pipeline diameter

The diameter depends on the type of concrete to be placed (max. aggregate, additives), the placing distance and the output. The "standard" pipeline diameter of 125 mm is adequate for a max. size aggregate of 32 mm and 1½''.

Reduction

The reduction pipes are the most loaded elements in a pipeline. They have therefore an appropriate wall thickness and are designed for extremely high pressures if super high pressure pumps are concerned. Only reduction pipes should be used which are purpose made for certain concrete pump models. Too short reductions increase the placing pressure with stiff concrete consistencies or result in blockages with difficult to pump concretes. Reductions act like fuses. Non-pumpable mixes and foreign matter automatically clog in the reductions.

Pipeline

Install new value pipes and, in case of need, thick-walled pipe sections directly after the concrete pump and also in places difficult of access. Use thin-walled pipe sections at the end of the pipeline. The pipeline should be carefully laid, firmly supported and well anchored.

Pipe cleaning head

The pipe cleaning head must be equipped with an easily readable gauge in good working order and a largely dimensioned air drain cock.

FIGURE IV.27

(3 of7)

Pipe connections

Tight pipe connections prevent concrete setting at the pipe joints. They facilitate the blowing or pushing out of the pipeline because neither compressed air nor pressurized water can escape.

Agitator

The agitator is no concrete mixer. It prevents the setting of concrete and destroys the formation of bridges across the suction apertures. When placing concrete with a max. size aggregate of 63 mm and 2½'' or 3'', the agitator shaft must be adapted to the respective requirements, e.g. by shortening the agitator paddles.

"Swallowing capacity"

The swallowing capacity of a pipeline is the max. concrete volume which can flow of itself through a descending pipeline. When using such a pipeline the output of the concrete pump must always be greater than the swallowing capacity of the pipeline so that the concrete column in the pipeline cannot break off.

Lubricating mix

It is a slurry in the consistency of K2 made of cement and water or of 2 parts of cement and one part of sand and water. It is only filled into the concrete pump as soon as the first regular mix has been prepared.

Rubber sponge ball

It serves for cleaning the pipeline. Soft rubber sponge balls are used for cleaning the placing booms of lorry-mounted concrete pumps. In this case cleaning is often performed by sucking the concrete back. Hard sponge rubber balls are used for blowing out with air or emptying long pipelines with water. Don't forget the paper plugs!

FIGURE IV.27

(4 of7)

Heavy-weight concrete

Heavy-weight concrete with a bulk density of more than 2.8 t/m^3 is pumpable. When using placing booms, the outreach possibly has to be reduced.

Clamping device

By means of a clamping device, the pipeline can easily be connected to the concrete pump. It enables simple disconnection of the pipeline from the concrete pump and consequently facilitates the dismantling of the reduction pipes.

Chippings

Concrete containing crushed material (chippings) is pumpable. The content of fines must be increased in accordance with the standards.

Relay pump

A relay pump should preferably be equipped with a strong, fast-running agitator for the purpose of "remixing" the concrete. A suitable communication system between the operating crews of the respective pumps should be provided.

Steel fibre concrete

Steel fibre concrete can be pumped successfully with SCHWING concrete pumps. Steel fibres must be filled into the mixer by means of a vibratory feeder.

Underwater concrete

Underwater concrete can be placed well with concrete pumps. Instead of a flexible end hose a rigid pipe section is used at the end of the pipeline which remains permanently immerged in the concrete. When changing the pouring points a shut-off valve operated by the concrete pump or a valve controlled by the concrete flow prevents the penetration of water into the pipeline.

FIGURE IV.27

Blockages

Blockages occuring with pumpable concrete are generally to be attributed to human failure. Blockages in or directly after the concrete pump (in the reductions) are announcing themselves by a rapid rise in pressure, blockages at the end of the pipeline by a slower rise in pressure!

Cause of blockages

Foreign matter in the pipeline: This can be prevented by using a grill in the charging hopper.

Bleeding concrete: Observe the mixing time, if necessary increase the content of fines or change the concrete composition by reducing the content of middle-sized grain.

When starting to pump: Wrongly composed lubricating mixes or too small quantities of lubricating mixes, dirty pipelines, formation of ice cakes in the pipeline in frosty weather.

Blockages immediately after the start of pumping: It can be concluded that the concrete in question is not pumpable. Unsuitable composition of aggregates, too small quantities of fines, inaccurate dosing of the aggregates, overflowing of aggregate bins, jammed cement scales, ignorance of the differing quantities of water contained in the sand and the coarse aggregates, inadequate mixing time, etc.

In case of repetitious blockages: Bleeding concrete, non-observance of the minimum mixing time requirements, too thin concrete consistencies when using crushed material or rough grain compositions, long pumping interruptions, changes in concrete consistencies.

When blowing out: Inadequate air quantity, insufficient air pressure, leaky pipelines, a soft instead of a hard rubber sponge ball, untight plug, blocked-up outlet, etc.

Vertical lines:

Vertical lines must be well anchored and accessible. Avoid swan-necks in a vertical pipeline. The lower bend to the vertical pipeline should be easy to dismantle and accessible for vehicles, if possible.

FIGURE IV.27

(6 of 7)

Water pressure device

All water pumps can be used as water pressure device which have a high conveying pressure (in excess of 25 bar) and a minimum output of 200 litres/min.

Brick chippings

Concrete containing brick chippings or other porous aggregates is pumpable. The aggregates must be pre-treated – watered. If necessary, increase the content of water.

FIGURE IV.27

(7 of 7)

--DEGRADATION OF COATINGS BY OZONE AND ULTRAVIOLET RAYS

Test	Crack width	Acrylic rubber type: Primer + base coat	Acrylic rubber type: Base + top coat	Polyurethane rubber type: Primer + base coat	Polyurethane rubber type: Base + top coat	Polybutadiene rubber type: Primer + base coat	Polybutadiene rubber type: Base + top coat
Exposed to ozone (5pphm x 68h.)	0.1mm					crack in base coat	
	0.3mm						break in top and base coat
	1.0mm						
	3.0mm				break in top coat	break	
Irradiated with ultraviolet rays (1000 h.)	0.1mm			cracks like a tortoise shell		cracks like stars	cracks in top coat
	0.3mm				cracks in top coat		
	1.0mm					cracks like stars	
	3.0mm			breaks in base coat	holes in base coat; breaks in top coat	point hole	

FIGURE IV.28 Degradation of Surface Coatings by Ozone and Ultraviolet Rays [51]

TYPES OF COATINGS

Coating	Primer	Base coat	Top coat
Highly elastic acrylic rubber	Synthetic resin containing epoxy	Acrylic rubber (water type) viscous slurry material	Acrylic urethane
Highly elastic polyurethane rubber	Epoxy resin	Polybutadiene rubber(solvent type) viscous slurry material	Acrylic urethan
Highly elastic polybutadiene rubber	Epoxy resin	Polyurethane rubber(solvent type) viscous slurry material	Acrylic urethane
Epoxy resin (for accelerated test only)	Epoxy resin	Epoxy resin (solvent type) viscous slurry material	Acrylic urethane
Thickness (μm) of membrane	0 – 30	1,000	100

FIGURE IV.29 Types of Surface Coatings and Their Layer Components [51]

SECTION V

CASE STUDY

BUILDING 64

NAVAL AIR STATION

ALAMEDA, CALIFORNIA

by

Max Rodgers

A. Introduction

The U.S. Navy has an ongoing underwater inspection program for it's shorefront facilities which is directed by the Ocean Engineering and Construction Project Office (FPO-1), Chesapeake Division, Naval Facilities Engineering Command (NAVFAC), Washington, D.C. The underwater inspection program is part of NAVFAC's Specialized Inspection Program. The program sponsors task-oriented engineering services for the inspection, structural analysis, repair recommendations and estimates of repair cost for the submerged portions of Naval Waterfront Facilities.

An inspection of Building 64 of the Alameda Naval Air Station was conducted during the period of January 23, 1990 through January 25, 1990 by Suboceanic Consultants, Inc. The specified objectives of this inspection were to conduct an onsite inspection in sufficient detail to assess the general structural condition of the designated facility and to document this assessment along with the inspection findings and recommendations in a formal engineering report. Appendix 6 describes the inspection procedures and terminology used to conduct the inspection.

As shown in Figure V.1, the Alameda Naval Air Station (NAS) is located on the eastern shore of San Francisco Bay. It is approximately five miles due east of downtown San Francisco, 10 miles east of the Golden Gate Bridge and about two miles southwest of downtown Oakland. The NAS occupies the Western one quarter of Alameda Island and is at the geographic center of the San Francisco Bay Area.

Water temperatures range from about 55 degrees (F) in the winter months to 63 degrees (F) in the summer. The average tidal range is 5.5 feet and the extreme range is 9.1 feet. Maximum tidal currents at the NAS waterfront are less than 1 foot per second. Water depths underneath building 64 range up to 10 feet and underwater visibility ranges about 4 feet but can be significantly influenced by tidal flows.

The existing waterfront facilities at the NAS include three berthing piers, a fishing pier, a fueling pier, an aircraft wave-off pier, two wharves, a concrete bulkhead with four seaplane ramps, two over water pile supported buildings (one of which is Building 64), recreational boat docks, two breakwaters, two jetties and extensive rock seawalls.

A map depicting the locations of the waterfront facilities located on the NAS is shown in Figure V.2.

Building 64 is located in the southeastern portion of the Seaplane Berthing Area beside Building 15 (see Figure V.2). It was constructed in 1941 and originally called "Carrier Pier Boiler House". Building 64 is presently used by the Ship Intermediate Maintenance Activity (SIMA) Divers. This inspection was the second underwater inspection of the building made under the current NAVFAC sponsored Underwater Inspection Program.

Building 64 is a two story concrete structure founded on 20 inch square reinforced concrete piles. As shown in Figure V.3, the building measures 42 feet 8 inches long by 30 feet 2 inches wide. The first floor slab is at elevation 113.0 feet (referenced to the base datum). Twenty-one piles supporting the structure are capped with reinforced concrete cap/footings. There are two piles per cap/footing, except at three locations, which have one pile each. The building is constructed over water and connected to land by two 30 foot long single span foot bridges.

B. Assessment

Piles supporting Building 64 were inspected in accordance with inspection criteria and procedures described in Appendix 6. All piles were given a Level I examination and Level II examinations were made on 5 piles as noted on Figure V.3 . Above water portions of the piles, cap/footings, beams and the underside of the floor slab were visually inspected from the water. Specific observations are tabulated in Table V.1 and summarized below.

Piles : Level I examination, below the mean low water elevation, revealed softening of the corner concrete on all 21 piles. Erosion of the soft concrete was apparent on four piles and reinforcing steel was exposed on two of the piles. When cleaned and sounded with a chipping hammer during Level II examinations, the soft corner concrete easily broke away to the reinforcing steel. The concrete surfaces in the central portions of the pile faces however, appeared firm and hard when struck with the hammer.

Above the mean low water elevation, cracking and spalling of the pile corners was observed on 14 of the piles. These deficiencies were all located from the pile cap down about 3 feet. Rust bleeding from the cracks and spalls indicated that

corrosion of the reinforcing steel caused the cracking and spalling. In spall areas, reinforcing was exposed by as little as 1 1/2 inch loss of concrete, which indicates insufficient concrete cover.

Photos V.1 through V.5 depict typical observations from the Level I and II examinations of the piles.

Pile Caps, Floor Beams and Deck Slab : The pile caps were all found to be in good condition. No deficiencies or anomalies of any kind were observed. With one exception, the floor beams were also found to be in good condition. The beam between the 2B and 3B piles had a short spall, about 3 inches wide by 3 inches deep, on the lower outboard edge. Reinforcing steel was exposed in the spall area. The underside of the deck slab was in good condition, except where holes had been drilled through for the floor drains of other purposes. Approximately eight drill holes were noted where concrete was broken away from the slab and the welded wire mesh reinforcing was exposed (see photo V.6).

Based upon the findings of the inspection, Building 64 was judged to be in good structural condition. The reinforced concrete piles supporting the building are undergoing progressive deterioration, below the mean low water elevation, as a result of

what appears to be sulfate attack. Above the mean low water elevation, progressive deterioration is also occurring. This deterioration is caused by corrosion of the reinforcing steel with subsequent spalling of the surface concrete. Thus far, the deterioration of the piles has not reduced their structural capacity below the design level. However, it was recommended that the piles be jacketed to prevent further deterioration.

The inspection firm, Suboceanic Consultants, Inc., recommended that all of the reinforced concrete piles supporting building 64 be jacketed from the pile capes to 2 feet below the mudline. It was also recommended that the areas of broken floor slab, where holes had been drilled through, be patched with Gunite.

Furthermore, it was recommended that Building 64 be reinspected immediately after the repair work is accomplished and at six year intervals thereafter.

1. Inspection Requirements

The initial requirements for the underwater inspection of Building 64 included providing the engineering services to document an underwater inspection and subsequently assess the integrity of the structural members supporting the facility. Assessment included the underwater inspection, engineering

analysis of existing conditions, review and comparison of previous inspection data and drawings of the facility, engineering calculations, recommendation of appropriate actions, and documentation of findings.

100% of the piles were to receive a Level I inspection, 10% of the piles were to receive a Level II inspection (at three different elevations) and 5% of the piles were to receive a Level III inspection. A minimum of 15 photos were to be included within the report and 15 copies of the final report was required.

2. Inspection Pattern/Procedure

The inspection was conducted by three engineer/divers. Two divers worked in the water and the third took notes. Observations were reported to the note keeper who tended the divers either from topside or in a small boat.

3. Inspection Equipment

SCUBA gear was utilized throughout the inspection because of the mobility it affords maneuvering around piles. Underwater photography equipment included Nikonos IV-A camera with 28 and 35 mm lenses and a Nikonos SB-101 underwater strobe. A clear water box was used for underwater photos because of the turbid water conditions present around Building 64. A small clear water

box was used with a subject frame of 5 inches by 7 inches. Above water, a Nikon 35 mm camera was used.

Chipping hammers, hand held scrapers, measuring tapes and calipers were used to clean and measure select piles. A small aluminum boat was used for topside support.

4. Estimate of Costs of Repairs

The required repairs to the piles supporting Building 64 include the jacketing of 21 concrete piles with reinforced concrete. The new concrete should extend from two feet below the mudline up to the pilecaps. The existing piles are 20 inches square and should be jacketed with a minimum of 6 inches of new concrete.

A cost estimate was prepared by Suboceanic Consultants, Inc. upon completion of their underwater inspection in the amount of $23,000.00. This price included all costs associated with material, equipment, labor, engineering and supervision as well as mobilization and demobilization cost.

C. Project Plan and Constraints

Due to the shrinking Department of Defense budget with subsequent reductions in all activities' budgets, it became obvious that funding for the repairs to the pilings supporting Building 64 were not going to become available in the near future so alternative measures were sought to effect the needed repairs.

An agreement was reached between the Naval Air Station, the tenants of Building 64 (SIMA), the San Francisco Public Works Center (PWC), the local Construction Battalion Unit (CBU-416) and the authors of this report (U.S. Naval Civil Engineer Corps Officers) in order to repair the building along with providing a case study for concrete repairs to port and harbor facilities. A cost estimate was prepared by the author in May of 1991 for materials and equipment/tools. This estimate totaled $7269.00 (see Table V.2) and was submitted to the Facilities Maintenance Officer of NAS Alameda with a request to complete the repairs as a self-help project by the tenants of the building.

The Naval Air Station provided $7000.00 in funding, the PWC provided the contracting vehicle to purchase the required materials, the CBU constructed the concrete forms, the SIMA

divers provided the labor and the authors provided the engineering support and supervision for the project.

Several constraints were imposed upon the repair project. These included an inflexible amount of funding, limited accessibility, man power availability and time constraints.

The $7000.00 made available by the Naval Air Station was a firm fixed amount and no additional funding would be available if the project suffered cost overruns. The location of the piles to be repaired (i.e. underneath a building and underwater) posed an accessibility problem which was overcome by utilization of a concrete pump and a 2" diameter hose which could be man handled beneath the building. The man power which would be available to conduct the repairs would be limited to their utilization on a strictly "not to interfere" basis from their assigned duties. And lastly, there was a time constraint to complete the project prior to the return of the NAS Alameda based ships from the Persian Gulf War due to the anticipated increased workload of the SIMA divers.

D. Concrete Mix Design

After considerable research of the available literature as well as discussions with engineers, professors and concrete professionals it was determined that a 3000 psi, low slump (4"-6"), low water/cement ratio (0.4) concrete utilizing 1000 pounds per cubic yard of pea gravel as course aggregate would be the appropriate concrete to use to jacket the piles.

One of the major constraints of the project was the location of the concrete piles, underneath an existing building as well as being submerged most of the time. Utilizing the chosen concrete mix design would allow the concrete to be pumped through a 2" diameter flexible hose which could be man handled underneath the building. Utilization of the 2" diameter hose also allowed the concrete to be placed underwater by tremie placement into the forms.

Durable concrete for the marine environment requires strict adherence to mix design criteria. A quality concrete mix must utilize a high strength, moderate tricalcium aluminate, uniform quality cement. Type II portland cement has proved to be highly successful in most marine applications. Aggregates must be

checked for soundness and purity with uniform grading in order to ensure stability at high slumps.

Modern admixtures are a necessity in order to obtain the workability while maintaining a low water cement ratio (0.4 - 0.45). The generous use of cement, 450 - 500 pounds per cubic yard, imparts a "self healing" quality to the concrete which can be beneficial in cracks and joints.

Efficient and consistent mixing is required in order to have a high quality concrete. This requirement is best achieved by the use of computer controlled batching plants. A print out of each batch allows for the high level of quality control and assurance that is required in order to insure durability of the concrete in the marine environment.

E. Project Execution

The repair project began in earnest in July 1991 once approval and funding was received. The CBU began constructing the forms, PWC issued the necessary contracts for the purchase of concrete and other required materials and the SIMA divers began the initial cleaning of the piles.

The piles contained an accumulation of 50 years of marine growth but they were all cleaned effectively and efficiently by using a saltwater "Hydro Blaster" adjusted to 10,000 psi delivery pressure. The cleaning of all 21 of the piles was completed within six hours with a work force consisting of six divers, rotating every half hour, and three top side personnel who controlled and monitored the Hydro Blaster compressor.

The reaction force from the nozzle of the Hydro Blaster was significant enough to require the operator/diver to secure himself to the pile by looping a 2 inch diameter line around both himself and the pile. An additional diver was required to position himself directly behind the nozzle operator in order to lend support for control of the Hydro Blaster nozzle. The on/off control of the nozzle was by top side support personnel who

constantly monitored the progress of the divers and received hand signals from the nozzle operator.

Once the cleaning of the piles was completed a through inspection of each pile was conducted to examine the surface preparation. All the piles were cleaned very well and the surface of the concrete was roughened enough to allow for good adhesion of the new concrete. The exposed reinforcing steel of the existing piles was effectively cleaned by the hydroblasting and no additional rebar was installed.

Upon completion of the cleaning operation, the original execution plan called for the piles to be coated with a bonding agent prior to installation of the welded wire fabric and subsequent placement of the forms and concrete. The practicalities of time and logistics soon demonstrated that this plan could not be followed within the allotted timeframe and budget. The actual execution of the project allowed for only three available times that the concrete pump would be on site and available for the placement of concrete. In order to carry out the optimal procedure of surface preparation, bonding agent application, installation of welded wire fabric, form erection and concrete placement in a continuous process would require an exorbitant amount of manpower as well as considerable amount of standby

time for the concrete batch plant and pump truck. This was completely impractical due to the small amount of concrete to be placed and the limited resources ($) with which we had to operate.

The decision was made to forego the application of the bonding agent. The roughness of the existing concrete surface coupled with the rich mixture of the new concrete was deemed to allow for sufficient bond strength between the existing and new concrete. Welded wire fabric was applied to seven piles and four forms were erected, three forms around double piles and one form around a single pile. The forms were leveled by jetting around their exterior bottoms with a fire hose and bracing them with timbers from the pilecaps of the existing piles. One inch wide banding straps were applied around the whalers of the forms as well as in between the whalers whenever sufficient space was available. Vertical 4x4 timbers were then banded into place along the exterior of the forms to provide additional stiffness to the forms.

The welded wire fabric installation and the form erection was completed in a single day and the forms were then filled with fresh water awaiting the placement of the new concrete the following morning.

It was discovered that the best procedure for form erection was to float the forms into place at high tide and level the forms as the tide went out and the forms began to settle into a resting position on the bottom. The installation of the banding was conducted at low tide in order to be able to install the bands on the lower whalers of the forms. This procedure proved to be effective however it required operations to be conducted as early as 3:00 AM and extend as late as 9:00 PM on some occasions. Without the use of military personnel as a labor force this requirement would has been a potential budget busting requirement (overtime).

Upon arrival and setup of the concrete pump truck, a two inch diameter discharge hose was led from the pump underneath the existing building to the forms. The pump and hose was primed with fresh water and discharged into the forms. The concrete was placed into the forms using a tremie technique in order to avoid the unnecessary mixing of salt water into the new concrete. The slump of the concrete was maintained between four and six inches which allowed for easy pumping as well as self-leveling of the concrete within the forms. Consolidation of the newly placed concrete was carried out by rodding with a one inch

diameter pipe. This consolidation method proved to be effective but was extremely difficult for the labor force.

The newly placed concrete was allowed to cure for seven days before the forms were removed, re-oiled and reset in the next location. Concrete placement occurred the next day after the forms were set and the procedure repeated itself for three iterations in order to repair all of the piles underneath Building 64.

Seven days after the final concrete placement, the forms were removed for the third and final time and discarded. The wooden forms had been exposed to the seawater environment so long that they were waterlogged and had no apparent salvage value.

One of the limitations placed upon the repair project, due to funding restrictions, was that the forms had to be constructed of a size that would allow them to be used at all locations underneath the building. This restriction resulted in the forms being constructed eight feet tall. This height allowed new concrete to be placed around the shallow piles almost all the way up to the pile cap. However, this restriction resulted in the deepest piles only receiving new concrete up to slightly above

the highest, high water level. A significant length of original pile is exposed above the new concrete on the deeper piles.

Above the new concrete on the deeper piles there is some deterioration due to the corrosion, cracking and spalling of the concrete cover. This deterioration was rectified by cleaning the corrosion products from the exposed reinforcing steel by hydroblasting as well as cleaning out any lose concrete from the cracks. Epoxy was applied to the exposed reinforcing and to the cracks in order to seal both from the detrimental marine environment.

The epoxy coating and filling completed the repair effort to the deteriorated concrete piles supporting Building 64. The base's Facility Maintenance Officer was given the details of the repair project and has initiated a monitoring system to track the durability of the repair.

Photos V.7 through V.19 are various photos documenting the repair effort to Building 64.

TABLE V.1

BUILDING 64 - OBSERVATIONS FROM LEVEL I EXAMINATION OF PILES

PILE	OBSERVATIONS
1A	Vertical crack 3/8" wide with rust bleeding out extends from cap down 3' on one corner. All corners soft below MLW.
1B (outboard)	One corner spalled (8"W x 1"D) from cap down 2.5'. Horizontal ties (reinforcing) is barely exposed in spall area. Crack, 1/8" wide, with rust bleedout extends from spall down 18". All corners soft below MLW.
1B (inboard)	All corners soft below MLW.
1C	Hairline horizontal crack across one face located 2' below cap. All corners soft below MLW.
2A (outboard)	Vertical crack 1/4" wide with rust bleeding out extends from cap down 2.5' on one corner of pile. All corners soft below MLW.
2A (inboard)	Vertical crack 1/4" wide with rust bleeding out extends from cap down 3' on one corner of pile. All corners soft below MLW. Soft concrete has eroded from one corner and 1 1/4 inch square longitudinal rebar is exposed.
2B (outboard)	Two vertical cracks 3/16" wide with rust bleeding out extend from cap down 18" on adjacent faces near corner. Spalling of corner concrete appears imminent. All corners soft below MLW.
2B (inboard)	Vertical crack 1/8" wide with rust bleeding out extends from cap down 2' on one corner. At 4' below cap two small pieces of wood are embedded in the concrete on one face. All corners are soft below MLW.
2C (outboard)	Two vertical cracks 1/8" wide with rust bleeding out extend from cap down 3' on adjacent pile faces near corner. Spalling of the corner concrete between cracks is beginning. All corners are soft below MLW.

TABLE V.1 (cont.)

BUILDING 64 - OBSERVATIONS FROM LEVEL I EXAMINATION OF PILES

PILE	OBSERVATIONS
2C (inboard)	All corners are soft below MLW.
3A (outboard)	Two vertical cracks 1/4" wide with rust bleeding out extend from cap down 3' on adjacent pile faces near corner. Spalling of the corner concrete between cracks is beginning. Isolated rust bleedouts on two adjacent faces 2' below cap. No visible cracking associated with rust bleedouts. All corners are soft below MLW. Soft concrete has eroded up to 2" deep from one corner.
3A (inboard)	Corner spalled (2 1/2"D x 3"W) from cap down 1' with reinforcing exposed and badly corroded in spall area. Vertical crack 1/4" wide extends from spall area down 18". Corner is beginning to spall where cracked. Vertical cracks 1/16" wide with light rust bleeding out near all corners from cap down 18". All corners are soft below MLW. Soft concrete is eroded 2 1/2" deep from one corner and reinforcing steel is exposed.
3B (outboard)	One corner spalled (12"W x 1"D) from cap down 18". No reinforcing exposed, but heavy rust stains. Vertical crack 3/16" wide with rust bleeding out extends 1' below spall area. All corners are soft below MLW.
3B (inboard)	Two vertical cracks 1/16" wide with rust bleeding out extend from cap down 2 1/2' on adjacent faces near corner. All corners are soft below MLW.
3C (outboard)	All corners are soft below MLW.
3C (inboard)	All corners are soft below MLW.
4A	Two vertical cracks (1/16" and 1/4" wide) with rust bleeding out extend from cap down 3' on adjacent pile faces near corner. Corner between cracks is beginning to spall. All corners are soft below MLW.

TABLE V.1 (cont.)

BUILDING 64 - OBSERVATIONS FROM LEVEL I EXAMINATION OF PILES

PILE	OBSERVATIONS
4B (outboard)	Two vertical cracks (1/16" and 1/4" wide) with rust bleeding out extend from cap down 3' on adjacent pile faces near corner. Corner between cracks is beginning to spall. All corners are soft below MLW.
4B (inboard)	Vertical cracks (up to 3/16" wide) with heavy rust bleedouts near two corners from cap down 2'. Corner concrete between cracks is beginning to spall. All corners are soft below MLW. Soft concrete has eroded up to 1 1/2" deep from one corner.
4C (outboard)	All corners are soft below MLW.
4C (inboard)	All corners are soft below MLW.

TABLE V.1 (cont.)

BUILDING 64 - OBSERVATIONS FROM LEVEL II EXAMINATION OF PILES

PILE	LOCATION	OBSERVATIONS
3C (inboard)	T	All concrete surfaces firm to hard.
	M	All concrete surfaces firm to hard.
	B	Three corners soft to a depth of 1 1/2". One corner eroded to depth of 2" with reinforcing steel exposed. Concrete in central portion of pile faces is firm to hard.
4B (inboard)	T	Vertical cracks with heavy rust bleedouts, on adjacent faces of two corners, extend from cap down 2'. Maximum crack width is about 3/16" and corner concrete is beginning to spall. All concrete surfaces are firm to hard.
	M	All concrete surfaces are firm to hard.
	B	All corners are soft to a depth of 2". When struck several times with a chipping hammer, two corners easily broke away exposing the corner reinforcing steel. Concrete in central portions of pile faces is firm to hard.

TABLE V.1 (cont.)

BUILDING 64 - OBSERVATIONS FROM LEVEL II EXAMINATION OF PILES

PILE	LOCATION	OBSERVATIONS
2B (inboard)	T	Vertical crack in one corner from cap down 3'. Crack width 1/4" with rust bleeding out. Concrete surfaces firm to hard.
	M	Concrete surfaces firm to hard. Two short pieces of 1x2 wood embedded in surface of concrete on one face.
	B	Corners concrete soft to 1" depth. Other surfaces firm to hard.
2C (inboard)	T	Concrete surfaces firm to hard.
	M	Concrete surfaces firm to hard.
	B	Three corners soft to 1" depth. One corner eroded to 2 1/2" depth with reinforcing steel exposed. Other surfaces firm to hard.
3A (inboard)	T	One corner spalled (3"W x 2 1/2"D) from cap down 12". Corner reinforcing exposed in spall and badly corroded. Three corners cracked from cap down 18". Cracks are 1/16" wide with rust bleeding out. All surfaces firm to hard.
	M	Open vertical crack extends from spall down 18" on corner of pile. Spalling of corner at this crack is imminent. Concrete surfaces are firm to hard.
	B	Three corners soft to 1 1/2" depth from mudline up 18". Soft concrete eroded 2 1/2" deep from one corner and reinforcing steel is exposed. Concrete in central portions of pile faces is firm to hard.

TABLE V.2

COST ESTIMATE FOR REPAIRS TO BUILDING 64

ALAMEDA NAVAL AIR STATION

Cost estimates based upon local vendor's phone quotes. Quantities based upon repairing 21 piles (3 singles and 9 doubles) by encasing with 8" additional concrete reinforced with 6" welded wire fabric using 3000 psi concrete in reusable wooden forms. Concrete placement will be by pumping.

CONCRETE : (3000 psi compressive strength @ 28 days)

single piles 3 cu yds x 3 piles = 9 cu yds

double piles 4.5 cu yds x 9 piles = 40.5 cu yds

total = 49.5 yds (say 50 cu yds)

50 cu yds x $70.00/cu yd = $3500.00

PLYWOOD : (4'x8'x1/2", construction grade)

single piles 6 sheets x 1 form = 6 sheets

double piles 9 sheets x 3 forms = 27 sheets

total = 33 sheets

33 sheets x $34.00/sheet = $1122.00

WHALERS : (2"X4"X8')

single pile 156 LF/pile x 1 form = 156 LF

double piles 221 LF/pile x 3 forms = 663 LF

total = 819 LF

819 LF x $0.31/LF = $254.00

WELDED WIRE FABRIC : (6"x 6" x 4', 50' rolls)

single piles 24'/pile x 3 piles = 72'

double piles 39'/pile x 9 piles = 351'

total = 423' (say 9 rolls)

9 rolls x $35.00/roll = $315.00

REINFORCING STANDOFFS :

single piles 24 standoffs/pile x 3 piles = 72 standoffs

double piles 36 standoffs/pile x 9 piles = 324 standoffs

total = 396 standoffs

396 standoffs x $2.00/standoff = $792.00

BONDING AGENT AND EPOXY :

single piles 0.5 gal/pile x 3 piles = 1.5 gal

double piles 1 gal/pile x 9 piles = 9 gal

total = 10.5 gal (say 11 gal)

11 gal x $16.00/gal = $176.00

FORM RELEASE AGENT :

single piles 0.5 gal/pile x 3 piles = 1.5 gal

double piles 1.0 gal/pile x 9 piles = 9 gal

total = 10.5 gal (say 11 gal)

11 gal x $10.00/gal = $110.00

CONCRETE PUMP :

3 placements x $200.00/placement = $600.00

SMALL TOOLS :

$400.00

SUMMARY :

CONCRETE	$3500.00
PLYWOOD	$1122.00
WHALERS	$254.00
WELDED WIRE FABRIC	$315.00
STANDOFFS	$792.00
BONDING AGENT	$176.00
FORM RELEASE AGENT	$110.00
CONCRETE PUMP	$600.00
SMALL TOOLS	$400.00
TOTAL	$7269.00

FIGURE V.1 VICINITY MAP, NAVAL AIR STATION

ALAMEDA, CALIFORNIA

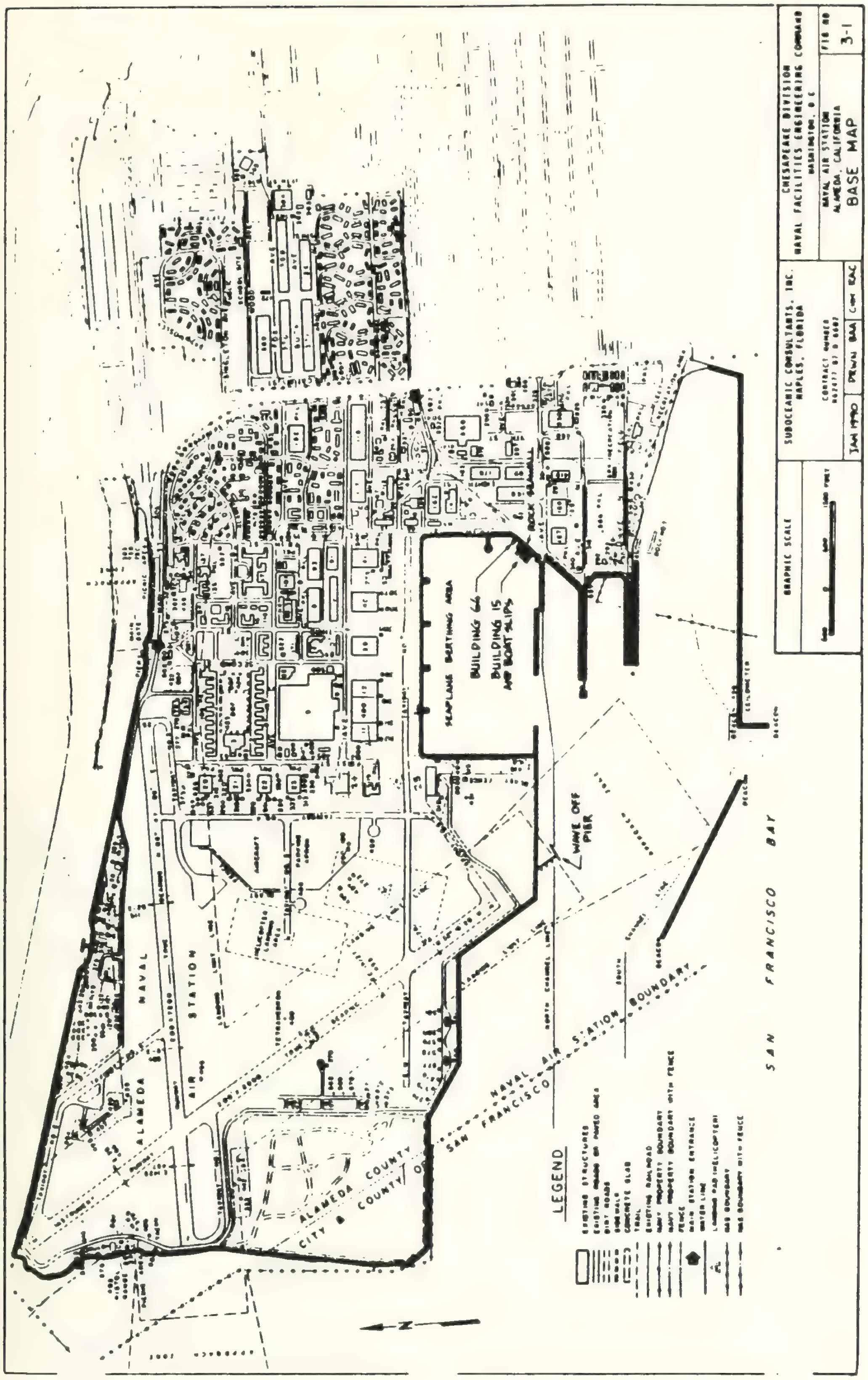


FIGURE V.2 BASE MAP, NAVAL AIR STATION

ALAMEDA, CALIFORNIA

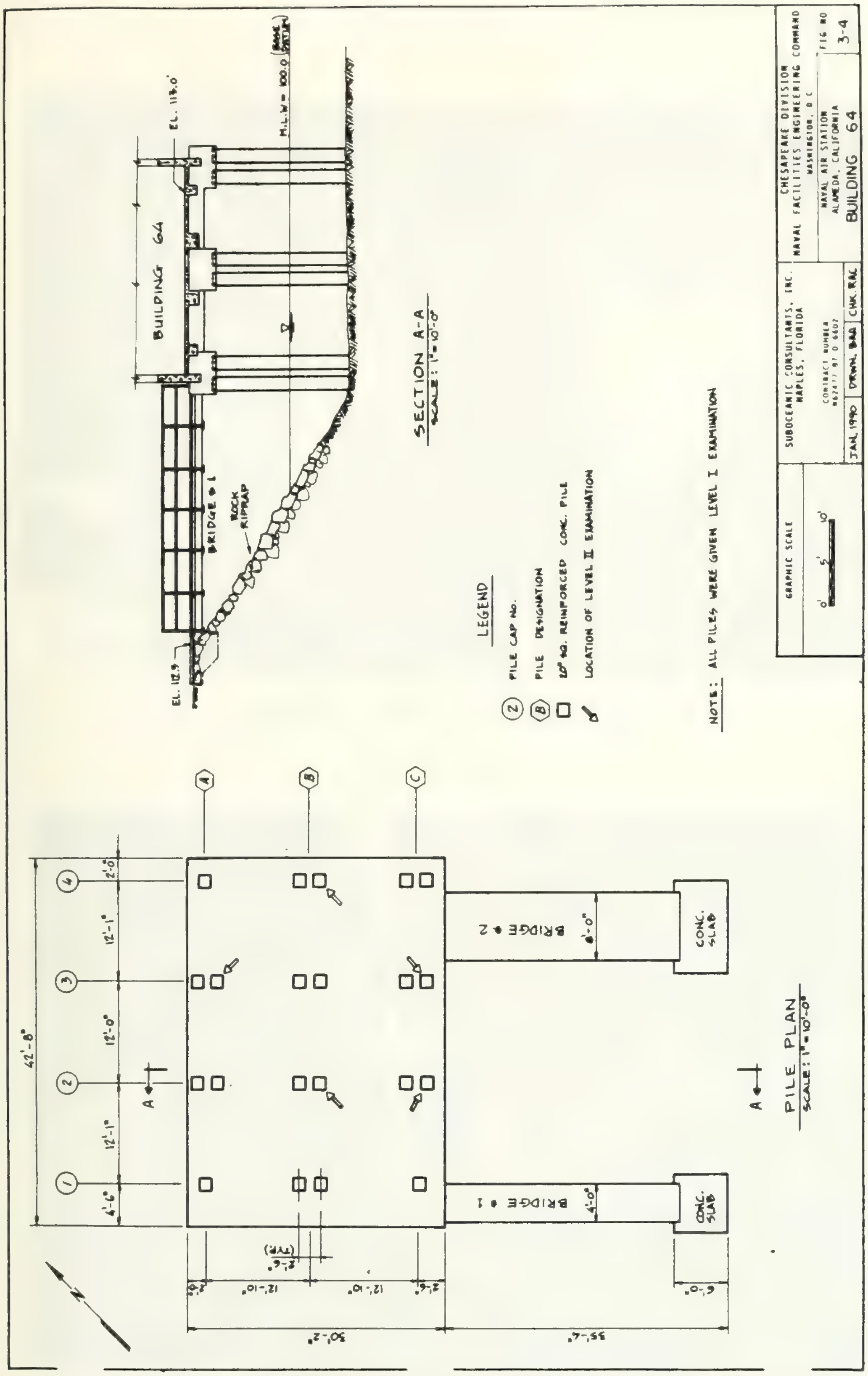


FIGURE V.3 BUILDING 64, NAVAL AIR STATION

ALAMEDA, CALIFORNIA

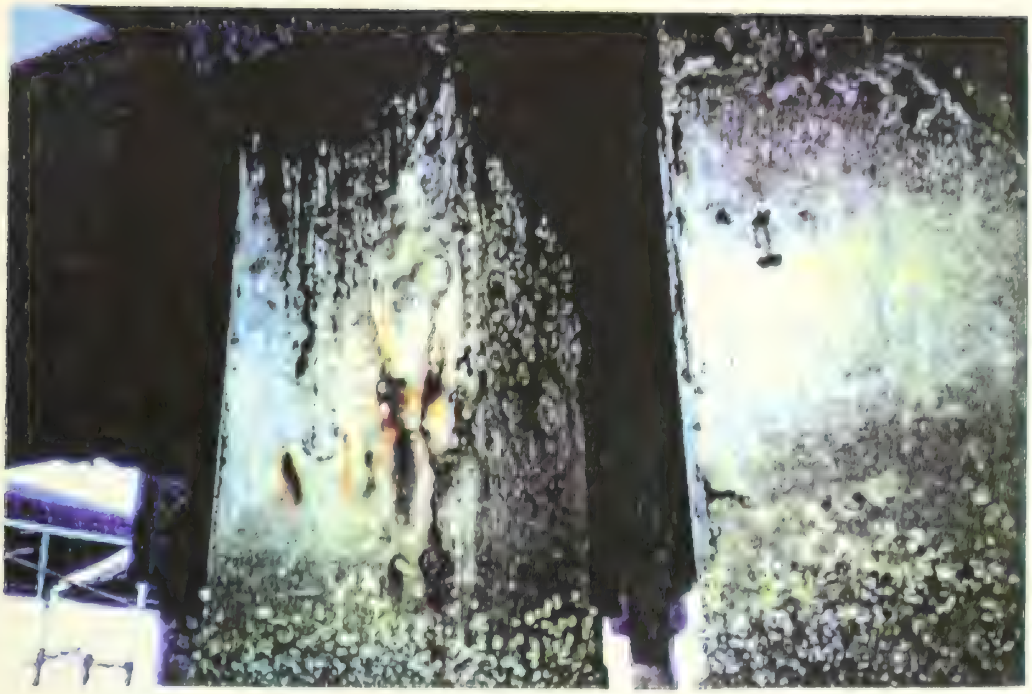


Photo V.1 Pile B1 (outboard)

Photo V.2 Pile A2 (inboard)

Photo V.3 Pile C2 (inboard)

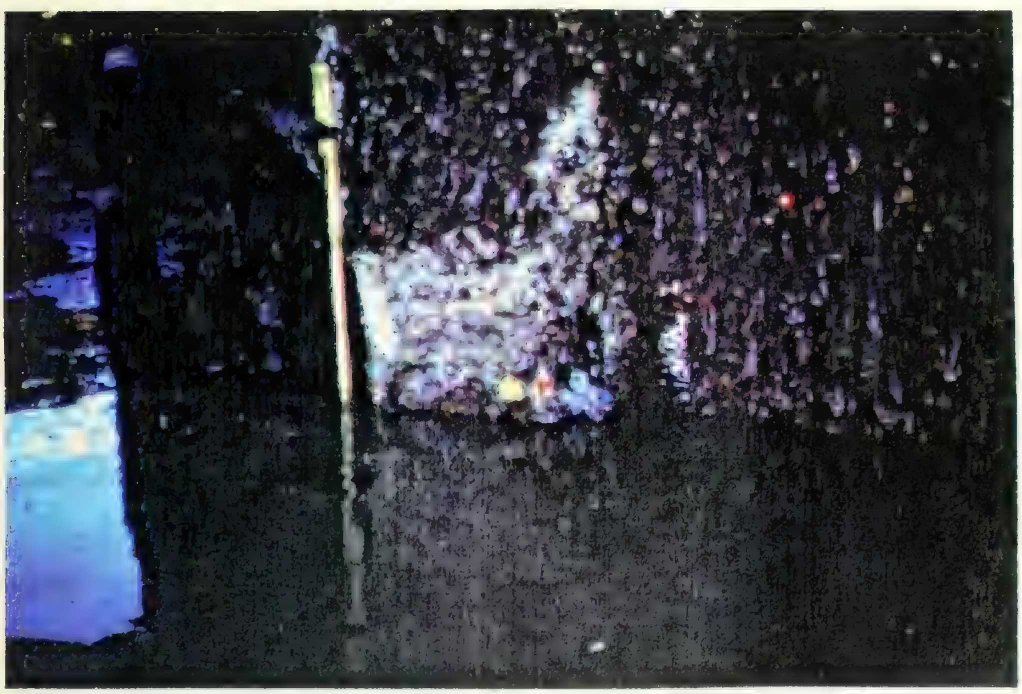


Photo V.4 Pile B4 (inboard)

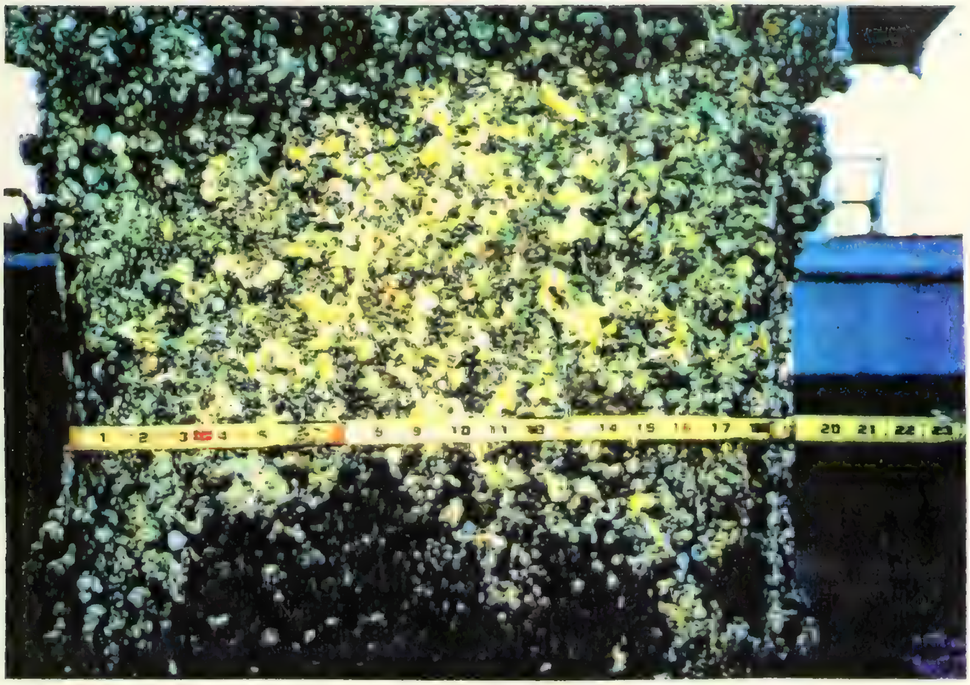


Photo V.5 Pile B4 (inboard)

Photo V.6 Bottom of floor slab

Photo V.7 Piles A2 cleaned and WWF installed

Photo V.8 Banding forms on piles A1 and B1

Photo V.9 Installing strongbacks for form on piles B1

Photo V.10 WWF installed on piles A3, form installed on Piles B3

and completed repair on piles C3

Photo V.11 Form inplace and braced on piles A2

Photo V.12 Forms installed on piles A1, A2 and A3

Photo V.13 Eight feet high forms and unusual high tide posed a problem before concrete placement on piles A1-A4

Photo V.14 Pump used to pump concrete from truck to forms

Photo V.15 Placing concrete in form around piles A2 at high tide

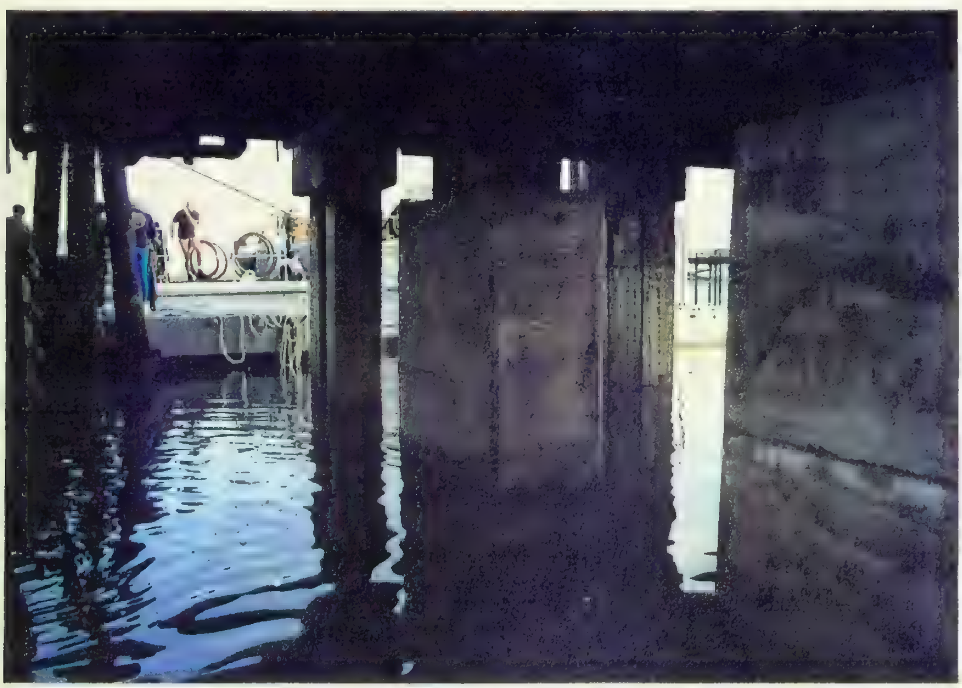


Photo V.16 Completed repairs on piles B2 and C2, WWF installed on

piles A3

Photo V.17 Completed repairs to piles C1-C4

Photo V.18 Completed repairs to piles A4, B4 and C4

Photo V.19 Completed repair to Pile A4

SECTION VI

EVALUATION OF REPAIR PROJECT

BUILDING 64, ALAMEDA NAVAL AIR STATION

by

Max Rodgers

A. Introduction

The repairs to Building 64 provided a good amount of experience to all those personnel who participated in the project. Numerous problems presented themselves but all were overcome and the project was a complete success. An evaluation of the repair effort can be best conducted by a discussion of the problem areas encountered and the resolution of those problems. The following section is a compilation of the problems encountered, the measures taken to overcome the problems and a noting of the lessons learned during the conduct of the repair project.

B. Problem Areas

1. Medical

One of the first problems to manifest itself was a discovery of the harmful effects upon the working personnel during the cleaning of the concrete piles. During the cleaning process a total of six personnel were utilized to clean the marine growth from the concrete piles supporting Building 64. The cleaning was accomplished by using a "Hydro Blaster" to remove the marine growth with a spray of salt water delivered at 10,000 psi. This method proved to be highly successful and cleaned the piles with the minimum amount of physical effort and in a timely manner.

The personnel operating the "Hydro Blaster" had to secure themselves to the pile they were cleaning by looping a 2 inch diameter rope around themselves and the pile in order to overcome the reaction force of the spray nozzle. This requirement placed the operators in a very close proximity to the pile they were cleaning. The danger of inflicting injury by the inadvertent directing of the salt water spray (10,000 psi) was readily apparent and proper precautions were utilized and no injuries were incurred.

As with numerous cases, the obvious danger was not the only danger present. The close proximity of the operating personnel to the discharge of the "Hydro Blaster" and the pile resulted in significant amount of the salt water spray and the marine growth coming in direct physical contact with the personnel. Operating personnel wore both facial protection as well as hearing protection during the entire cleaning process yet each and every one were stricken with a severe case of diarrhea which persisted for up to 48 hours after the cleaning phase of the project.

All personnel were treated at the station medical facility and all had fully recovered within 48 hours. Participation in the cleaning of the concrete piles with the "Hydro Blaster" was the only item which each and every stricken person had in common and the opinion of the local medical staff was that inadvertent ingestion of some type of marine organisms from the cleaning process was the likely cause of the diarrhea.

Future operations of this nature should be conducted only with the personnel involved being required to wear respirators which will filter the inspired air and eliminate the possibility of inadvertent ingestion of marine organisms.

2. Form size and weight

The necessity to man handle the forms underneath Building 64 produced another problem. Inaccessibility due to being underneath an over water building required the movement of the forms to be done by man power. The forms were eight feet high, sixty-four inches by thirty-two inches wide and constructed of one half inch plywood and two by four whalers. The forms weighed about 500 pounds each and required a minimum of six men to move each one.

It was impossible to reach underneath the building with any available equipment capable of lifting the forms, therefore each move required a considerable amount of effort. The problem of the excessive weight was quickly overcome by planning the form-setting operation around the proper tidal occurrence. Being constructed of wood, the forms possessed a significant amount of buoyancy and at high tide, the forms could be floated into place and submerged by personnel climbing up on the forms and forcing them to rest on the bottom while strong backs were positioned to hold the forms securely into place.

This evolution became smoother with each iteration and the size and weight of the forms did not pose a significant problem as long as the tides were used to our advantage.

3. Tidal range

Besides relocating the forms, other work required attention to the tide underneath Building 64. In order to secure the bottom of the forms it was necessary to band around the whalers with one inch wide banding straps. This operation could not be efficiently conducted underwater, therefore the banding operation was conducted at the lowest tide each day. This requirement lead to the necessitity of having to begin work at 3:00 AM on several occasions as well as having to extend operations until as late as 9:00 PM on some occasions.

Even though the early and late hours were not entirely popular, all required personnel were available as needed and the work was accomplished without any significant problems.

4. Resource optimization

Due to the fixed and limited amount of money available and the continued availability of a work force being uncertain, optimization of manpower and resources had to be planned thoroughly in order to avoid any inefficiencies in the repair operation. By working closely with and maintaining constant communications with all involved parties, including the Sea Bee Unit, the Public Works Center, the SIMA dive locker and the

material suppliers, the repair project was completed on time and within budget without any obvious waste.

5. Flexible tremie hose

The necessity to tremie the concrete through a two inch diameter flexible hose proved to be a minor problem. A flexible hose was required for the pumping of the concrete in order to reach underneath Building 64. The flexible hose was feed down into each form and as the level of the concrete rose within the form, the nozzle of the hose tried to float out of the concrete. This buoyant force had to be overcome by continuous pressure being applied to the hose by a person. The quantity of concrete required to fill each form required this opposing pressure to be maintained for long periods of time.

The buoyant force demonstrated why tremie pipes are usually rigid however, the location of the forms prevented the utilization of a rigid tremie pipe. The problem was overcome by brute force (i.e., a Navy deep sea diver can overcome the head pressure of concrete being pumped through a two inch hose seventy feet long).

6. Sealing of form bottoms

A fairly significant problem was encounter when the bottom of one of the forms was blown out by the incoming concrete. An inspection of the harbor bottom underneath Building 64 by the author resulted in the forgoing of bottom closures being installed on the forms. The firm bottom appeared sufficient to withstand the head-pressure of the concrete to be placed in the forms.

This assumption was found to be true except for one occasion when the nozzle of the flexible hose was inserted too far into the form and was actually forced into the exterior corner of one of the forms. The pressure of the concrete was sufficient to displace the soil from underneath the edge of the form and the concrete flowed outside of the form. This problem became readily obvious and placement into that form was terminated.

The region adjacent to the exterior of the form was jetted with a fire hose in order to wash any concrete from around the bottom edge of the form. This procedure proved to be only partially effective. Upon subsequent completion of the concrete placement it was discovered that the form was concreted into place and a significant effort was required to break the form free.

The removal was successful with only minimal damage to the form which was repaired and used again. Greater attention was required with regards to the amount of flexible hose inserted into the forms and no further incidents of blown forms occurred during the remainder of the repair project.

7. Consolidation of the new concrete

Complete consolidation of concrete is one of the requirements of durable marine concrete and this requirement proved to be a difficulty. The requirement to have numerous personnel positioned within the water near the concrete placement eliminated the possibility of utilizing an electric vibrator. The possibility of electrical shock was too great and therefore consolidation was accomplished by rodding the fresh concrete during placement within the form with a two inch diameter steel pipe.

Utilizing the pipe for consolidation proved to be adequate yet very laborious. The restricted available space around the top of the forms proved to be the most difficult hurtle to overcome during the consolidation of the concrete. Once again perseverance and brute strength overcame adversity and the

concrete was consolidated adequately. No honeycombing was noticed upon subsequent removal of each of the forms.

8. Form strength

Despite the enormous size of the forms, there was evidence of inadequate form strength during the first concrete placement. The strain imposed into the forms was very evident and the placement was required to proceed at a vary slow pace in order to prevent the loss of any one of the forms.

No forms were lost but additional precautions were taken on all subsequent placements. Additional numbers of banding straps were applied as well as the addition of vertical stiffeners being attached to the exteriors of the forms. These additional vertical stiffeners consisted of two four by four timbers securely banded to each side of the forms. This additional stiffness proved to be adequate and relieved any concerns for personnel safety working around the forms during the concrete placement.

9. Labor force training

The inexperience of the labor force (i.e., the divers) proved to be a small problem with regards to initial scheduling. An obvious learning curve was apparent but each subsequent operation went more smoothly than the last and familiarity with the required

amount of work resulted in a much increased work output efficiency.

The amount of time and effort required for form removal, repositioning, securing and concrete placement dropped with each subsequent evolution. The required amount of time to wreck, reposition and secure each form dropped from an initial 24 man-hours to a best time of 12 man-hours for the last placement.

C. Conclusion

The repairs to Building 64 were completed on time and on budget without injury to any personnel. The intent of the repair effort was to stop the deterioration of the concrete piles due to corrosion of the reinforcing steel. The effectiveness of the repair effort can only be judged after a significant period of time has passed.

The repair techniques utilized for this repair were very unsophisticated and were carried out by inexperienced personnel with a great deal of efficiency. This method of concrete pile repair can be employed wherever future situations may arise. Attention should be paid to the lessons learned from this project and success can be achieved on future similar projects.

The problems identified during this repair are summarized below:

1. Divers contacted diarrhea after cleaning operation.
2. Man handling the forms due to size and weight.
3. Necessary to work at low tide.
4. Optimization of manpower and resources.
5. Tremie concrete thru a flex hose.
6. Blowing out bottom of forms.

7. Difficulty in consolidating concrete within forms

8. Adequate form strength.

9. Inexperience of labor force

SECTION VII

INTERVIEWS CONCERNING REPAIRS TO DETERIORATED CONCRETE PORT AND HARBOR STRUCTURES

by

Max Rodgers

A. Introduction

An exhaustive search of the available literature has revealed that there is very little published information which directly addresses repairs to concrete port and harbor structures. Specific information with regards to repair techniques, materials selection and operational procedures is at best, difficult to locate, if available at all.

In an effort to acquire information directly related to techniques, materials and procedures, a series of questions was developed by the author and presented to recognized professionals in the field of concrete repairs who have experience directly related to concrete port and harbor structures.

Interviews were conducted with Mr. Fred Aichele, Mr. Harvey Haynes and Professor Emeritus Ben C. Gerwick, Jr.. Mr. Aichele is the owner of Inshore Divers, Inc. of Pittsburg, California and his company specializes in the underwater repair of concrete port and harbor structures with major emphasis on pile restoration. Mr. Haynes is the president of an engineering consulting firm who specializes in the repair to concrete

structures. Mr. Haynes major field of expertise is in deterioration due to delamination and is therefore interested in the durability of concrete repairs. Professor Gerwick is Chairman of the Board of Ben C. Gerwick, Inc., an engineering consulting firm which specializes in the construction of marine concrete structures.

The interviews were conducted by the author and the major points will be summarized within the conclusion of this section. The transcriptions are not meant to be direct quotations but rather are accurate presentations of the information provided for each question.

Appendix 7 contains two papers entitled Restoration and Preservation of Marine Structures by Divers and Modern Inspection Techniques in Port Maintenance. These two papers were coauthored by Mr. Aichele and provide some valuable information applicable to repairs of concrete port and harbor structures.

B. INTERVIEW WITH MR. FRED AICHELE

Date : 6 SEP 91

I. General

1. Please describe your background and give a brief biographical sketch of your education and professional career.

22 years US Navy, Retired Master Diver, One of original members of Harbor Clearance Unit, Experience in salvage, experimental diving and deep sea diving, Sole proprietor of Inshore Divers, Authored several papers on underwater restoration of port and harbor strictures.

2. How many years experience do you have in the repair of concrete port and harbor facilities?

8 years.

3. Based upon your experience, how effective and durable are current concrete repairs to port and harbor structures?

Very effective - major problem is that piles are only repaired from the waterline down about 6 - 8 feet vice all the way to the mudline.
10 - 15 year durability depending on the method used and the capability of the repair personnel.

4. Are most repair procedures for concrete port and harbor facilities aimed at restoring structural capacity or simply deterioration abatement?

Repairs directed at deterioration above the waterline.
"Out of sight - Out of mind" mentality exists.

5. What is the most important factor to consider when planning a concrete repair for a port or harbor facility?

Logistic support.
The work is easy compared to the effort to get to the work site with all the equipment and personnel.

6. How effective are current repair techniques in restoring capacity and durability of concrete port and harbor facilities?

Tough to get acceptance for new methods.
Engineer has own mind set regarding techniques and is not open to new ideas suggestions.

7. Are there any special considerations that have to be given to concrete being utilized in a repair situation as opposed to a typical concrete placement?

Use 8 sack mix, 2 hour batch life, 4000 psi (28 days), use gas piston pump, 7 -8 inch slump for pumpability and filling of void spaces.

8.Do you have any experience in comparing the durability of tremie concrete to conventional concrete placement with regards to port and harbor facilities?

All work is either tremie or pumped concrete.

9. Do underwater repair techniques differ substantially from above water repair techniques? If so, what are the major differences.

No, anything that can be done above water can be done underwater but it will be from 2 to 2.5 times more difficult and costly.

II. Assessment

10. What is the best technique to identify deteriorated concrete port and harbor structures, both above and below the waterline?

Visual is best method.

Hammer for soundness.

Biggest problem is not being tasked to clean properly.

11. How much of an adverse effect does the lack of accessibility pose when planning an in place repair to a port or harbor facility?

No problem for inspections.

Fender systems pose biggest problem for repair work.

12. How do you judge the effectiveness of a concrete repair to a port or harbor facility?

Swim by inspection after work is complete.

13. Do the techniques used for estimating the cost of repairs differ substantially for port and harbor structures as opposed to terrestrial structures? If so, please elaborate.

No.

14. What environmental considerations need to be taken into account when planning a repair procedure for a port or harbor facility?

Must make provisions to trap concrete, i.e., filter cloth, install pollution boom, pump off surface of water.

III. Procedures

15. In your opinion, what is the most effective technique to repair concrete piles in place?

Fiberglass/epoxy wrap.

16. What is the most effective way of repairing the concrete decks of piers and wharfs?

N/A

17. What are the major steps involved with a typical repair procedure for port and harbor facilities?

Survey, takeoffs, mobilization, conduct repair and demobilization.

18. Does a particular concrete mix appear to be more effective as a repair material than others for port and harbor facilities?

Use rich mix - 8 sack per yard.

19. What do you consider to be the most important parameter of the mix design when planning a concrete repair for a port or harbor facility?

Type and amount of retardant and plastisizers.
Slump (permeability).

20. How much curing time do you recommend for repaired concrete port and harbor structures?

As soon as 24 hours after placement.

21. What are to best curing techniques to employ for port and harbor facilities?

Water curing.

22. What type of field tests should be conducted during the repair procedure for concrete port and harbor facilities?

Cylinders for compressive tests.
Slump test.

23. What type of forms are available for port and harbor structures on the market today?

Fiberglass, wood, sheet metal, nylon, polyethylene & PVC.

IV. The Future

24. Is the current available literature sufficient for facility managers to be able to make responsible decisions with regards to required repairs to their concrete port and harbor facilities?

Very little exists on UW repair techniques.

Pile restoration is not sophisticated.

25. What areas of technology need to be expanded to produce better concrete repairs to port and harbor facilities?

Materials and techniques for concrete restoration.

26. Do you have any additional comments you would like to make with regards to the conduct of concrete repairs to port and harbor structures?

Need standardization of reinforcing requirements.

Need standard techniques.

Need to get rid of the out-of-sight-out-of-mind mentality.

C. INTERVIEW WITH MR. HARVEY HAYNES

Date : 20 September 1991

I. General

1. Please describe your background and give a brief biographical sketch of your education and professional career.

BSCE Clarkson University 1965, MSCE University of Washington 1966, 15 years at Naval Civil Engineering Laboratory, Professional Civil Engineer, founded engineering consulting firm in 1982 specializing in concrete consulting services, author of numerous papers and teaches and conducts seminars on concrete construction for the University of California at Berkeley.

2. How many years experience do you have in the repair of concrete port and harbor facilities?

8 years (25 years total in concrete work).

3. Based upon your experience, how effective and durable are current concrete repairs to port and harbor structures?

Depends upon workmanship.

Can be durable if done properly.

4. Are most repair procedures for concrete port and harbor facilities aimed at restoring structural capacity or simply deterioration abatement?

Indeterminate, not enough information available to make a judgement.

5. What is the most important factor to consider when planning a concrete repair for a port or harbor facility?

Proper specifications and qualified personnel.

6. How effective are current repair techniques in restoring capacity and durability of concrete port and harbor facilities?

Dependent upon quality of workmanship.

7. Are there any special considerations that have to be given to concrete being utilized in a repair situation as opposed to a typical concrete placement?

Surface preparation.

Compatibility of materials (i.e. thermal coefficients).

Shrinkage characteristics.

8.Do you have any experience in comparing the durability of tremie concrete to conventional concrete placement with regards to port and harbor facilities?

Only option is tremie or pumping.

9. Do underwater repair techniques differ substantially from above water repair techniques? If so, what are the major differences.

NA

II. Assessment

10. What is the best technique to identify deteriorated concrete port and harbor structures, both above and below the waterline?

Visual.

Strike with a hammer.

Core and conduct analysis.

11. How much of an adverse effect does the lack of accessibility pose when planning an in place repair to a port or harbor facility?

Increases the cost and the mobilization effort.

12. How do you judge the effectiveness of a concrete repair to a port or harbor facility?

Bond test - tension test.

Develop criteria for passing.

Determine mechanism of delamination.

13. Do the techniques used for estimating the cost of repairs differ substantially for port and harbor structures as opposed to terrestrial structures? If so, please elaborate.

Tough to do - too many unknowns.

A large data base does not exist.

14. What environmental considerations need to be taken into account when planning a repair procedure for a port or harbor facility?

Cleaning procedures.

Can't let anything go into the water.

Need permission up front - permits.

III. Procedures

15. In your opinion, what is the most effective technique to repair concrete piles in place?

NA

16. What is the most effective way of repairing the concrete decks of piers and wharfs?

Indeterminate.

17. What are the major steps involved with a typical repair procedure for port and harbor facilities?

Development of specifications and criteria, surface preparation, repair, clean up and testing.

18. Does a particular concrete mix appear to be more effective as a repair material than others for port and harbor facilities?

High cement content (but be aware of shrinkage if used above water), low W/C ratio, high impermeability, use silica fume.

19. What do you consider to be the most important parameter of the mix design when planning a concrete repair for a port or harbor facility?

Specifications.

20. How much curing time do you recommend for repaired concrete port and harbor structures?

Depends upon job location and type of repair.

21. What are to best curing techniques to employ for port and harbor facilities?

Provide shade and curing compound should be used.

22. What type of field tests should be conducted during the repair procedure for concrete port and harbor facilities?

Pull off test (not conventional).

23. What type of forms are available for port and harbor structures on the market today?

NA

IV. The Future

24. Is the current available literature sufficient for facility managers to be able to make responsible decisions with regards to required repairs to their concrete port and harbor facilities?

Needs more work.

All that exists today is case histories.

25. What areas of technology need to be expanded to produce better concrete repairs to port and harbor facilities?

Understanding of the delamination process.

Better surface preparation techniques addressing time element.

Compatibility of materials.

Minimize shrinkage (i.e. prepacked repairs).

26. Do you have any additional comments you would like to make with regards to the conduct of concrete repairs to port and harbor structures?

Need to address the whole system, i.e. workmanship, special attention for marine applications, develop guide specs, train special inspectors and give added authority to personnel responsible for quality control.

D. INTERVIEW WITH MR. BEN C. GERWICK, JR.

Date : 25 OCT 91

I. General

1. Please describe your background and give a brief biographical sketch of your education and professional career.

Graduated University of California at Berkeley in 1940 with a BS in Civil Engineering, 5 years with US Navy (at sea), 25 years in marine construction (mainly port and harbor structures), last 10 years of marine construction spent extensively in Middle East and Southeast Asia.

2. How many years experience do you have in the repair of concrete port and harbor facilities?

40 years (intermittently).

3. Based upon your experience, how effective and durable are current concrete repairs to port and harbor structures?

Range from excellent to very poor, the majority of the repairs are satisfactory but a significant number have subsequent corrosion problems.

4. Are most repair procedures for concrete port and harbor facilities aimed at restoring structural capacity or simply deterioration abatement?

Deterioration abatement.

5. What is the most important factor to consider when planning a concrete repair for a port or harbor facility?

The behavior of the repair in relationship to the original structure. The repair must be integrated into the original structure.

6. How effective are current repair techniques in restoring capacity and durability of concrete port and harbor facilities?

Very effective in restoring capacity. Durability is variable.

7. Are there any special considerations that have to be given to concrete being utilized in a repair situation as opposed to a typical concrete placement?

Bond properties of the repair material, use fine aggregate (3/8" max), placability, material cannot be allowed to sag or run.

8.Do you have any experience in comparing the durability of tremie concrete to conventional concrete placement with regards to port and harbor facilities?

Tremie concrete has been successful because it is placed underwater.

9. Do underwater repair techniques differ substantially from above water repair techniques? If so, what are the major differences.

Yes, surface preparation is more difficult, dimensional control and placability. Reinforcing steel must receive proper preparation.

II. Assessment

10. What is the best technique to identify deteriorated concrete port and harbor structures, both above and below the waterline?

Underwater - visual and hand held hammer.
Above water - half cell, linear polarization, infrared and ultrasonic as well as visual and hand held hammer.

11. How much of an adverse effect does the lack of accessibility pose when planning an in place repair to a port or harbor facility?

Definite problem underneath piers and wharves (lack of headroom).

12. How do you judge the effectiveness of a concrete repair to a port or harbor facility?

Visual and NDT techniques being especially watchful around the edge of the repair.

13. Do the techniques used for estimating the cost of repairs differ substantially for port and harbor structures as opposed to terrestrial structures? If so, please elaborate.

Yes, due to difficulty in accessibility and having to work underwater. Contamination of surface is a problem due to algae growth with time.

14. What environmental considerations need to be taken into account when planning a repair procedure for a port or harbor facility?

Keep both deteriorated material and new material out of the water.

III. Procedures

15. In your opinion, what is the most effective technique to repair concrete piles in place?

Jacket with fiberglass and cement grout around the annulus of the pile. Use sacrificial anodes if underwater.

16. What is the most effective way of repairing the concrete decks of piers and wharfs?

Chipping out deteriorated material, clean steel completely, wrap ends of reinforcing steel with zinc wire, apply epoxy coating and cover with shotcrete.

17. What are the major steps involved with a typical repair procedure for port and harbor facilities?

(Same as above)

18. Does a particular concrete mix appear to be more effective as a repair material than others for port and harbor facilities?

Use small aggregate, rich mix, same water/cement as original mix and don't use super plastisizers.

19. What do you consider to be the most important parameter of the mix design when planning a concrete repair for a port or harbor facility?

Impermeability.

20. How much curing time do you recommend for repaired concrete port and harbor structures?

Variable.

21. What are to best curing techniques to employ for port and harbor facilities?

Membrane curing (using epoxy coating).

22. What type of field tests should be conducted during the repair procedure for concrete port and harbor facilities?

Obtain cylinders for strength tests and cubes if shotcrete.

23. What type of forms are available for port and harbor structures on the market today?

Fiberglass, Steel, Plastic (PVC).

IV. The Future

24. Is the current available literature sufficient for facility managers to be able to make responsible decisions with regards to required repairs to their concrete port and harbor facilities?

No, a consist and complete commentary needs to be published directed towards repair.

25. What areas of technology need to be expanded to produce better concrete repairs to port and harbor facilities?

Providing cathodic protection for the splash zone.

26. Do you have any additional comments you would like to make with regards to the conduct of concrete repairs to port and harbor structures?

There still exists the question as to whether it is better to epoxy coat reinforcing steel or leave it bear. Work needs to be done in this area to settle this issue.
Epoxy coatings should be utilized more frequently. This is based upon the fact that the coatings applied to North Sea platforms 15 years ago as a curing compound have performed well to date

and also the "impermeable concrete" in the Middle East with an epoxy coating has no chloride intrusion whereas the uncoated surfaces are at threshold levels.

E. Conclusion

As a result of conducting the preceding interviews, the author gained considerable insight into the state of the technology and practice with regards to repair of concrete port and harbor structures. Several important points should be mentioned which appeared within the interviews as either common themes or as important areas which should be addressed when considering the repair to a concrete port or harbor structure.

The universal opinion among all of the interviewees is that the effectiveness of a repair to a concrete port or harbor structure is directly dependent upon the workmanship and environmental conditions present at the time. The effectiveness of a repair is determined by the durability of the repair and in order to obtain the required durability quality must be built into the repair.

A particular point which the author found interesting was the importance placed upon different aspects of a repair by the interviewees. Mr. Aichele is involved with performing the repair and to him the logistics associated with getting men and material to the worksite ranked supreme. Mr. Haynes is involved with the

engineering aspects of the repair material and his primary concern is with the specifications and quality of the personnel. Mr. Gerwick's perspective as a contractor/engineer is slanted toward considering how the repair will effect the entire structure. Each area of interest and level importance is justifiable and should all be considered when faced with the requirement to repair a concrete port or harbor structure.

Irrespective of the background or area of interest, there was a consensus among the interviewees with respect to the repair mix. A rich mix (7-8 sacks of cement per cubic yard of concrete) which is pumpable and compatible with the existing structure should be utilized after proper surface and reinforcing preparation. Attention should be given to the bondability of the repair material to the existing structure. Mr. Haynes recommended the use of an unconventional "pull-out" test to be used on the repair material to ensure proper bonding of the material to the existing material. Mr. Gerwick points out the difficulty in underwater surface and reinforcing preparation as well as dimensional control of the repair material.

A consensus also existed with regards to the best assessment technique to use. Visual inspection and sounding with a hand held hammer appears to be the technique of choice and gives

reliable and repeatable results. The interviewees also were in agreement with regards to the use of fiberglass wraps for the repair of deteriorated concrete piles.

Each interviewee agreed that environmental concerns are becoming a larger issue with regards to repairs to concrete port and harbor structures just as with new construction. The acceptable remedy appears to ensure that none of the deteriorated material nor the repair material enters the surrounding water. This can only be accomplished by booms and other containment devices which are submitted and acceptable to the controlling agency.

Lastly, a consensus also exists between all of the interviewees with regards to the need for additional information in the literature. Mr. Aichele expressed the desire to see some standardization of techniques and specification for repair procedures and materials. Mr. Haynes recommends that the whole repair system be addressed from the specifications for the materials, the training of the personnel to the techniques to ensure proper and adequate quality control. Mr. Gerwick suggested that additional work be carried out to investigate the validity of epoxy coating of the new rebar as well and the

utilization of a coating of all the concrete surfaces located within the splash zone.

Each of the interviewees are successful in their chosen profession and have considerable experience in the repairs to concrete port and harbor structures. It will be beneficial to heed the words of advice which each has graciously allowed to become a section of this report and to continue in the development of the materials and procedures to enable effective and efficient repairs to concrete port and harbor structures.

SECTION VIII

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE STUDY

by

Jim Schofield and Max Rodgers

"Quality is job one", exclaims the advertisement of one of America's largest automakers. If the results of this study were to be summarized in four words, that automaker's motto would serve well.

Repairs to concrete port and harbor structures rely heavily on quality, perhaps even more so than new construction. Quality materials, repair procedures, and workmanship are essential to successful repair projects as seen in the foregoing pages.

A clear understanding of the marine environment and its relationship to causes of deterioration of concrete is essential to the repair process. For example, interrelationships exist between repair materials and methods and the environment in which they are to be applied. Similarly, assessment methodologies are to some extent selected based upon the types of deterioration encountered during macro-level inspections. The interdependencies continue as assessments lead to the selection of repair methods, and it becomes clear that effective repairs cannot be performed without a complete understanding of the marine concrete milieu.

This study has reaffirmed that many of the fundamental environmental factors, deterioration modes, and repair materials

lead to relatively low technology repair methods that are quite effective when applied in a high quality manner.

The case study at the Naval Air Station Alameda served to demonstrate in actual practice for the authors many of the environmental influences, repair material placement difficulties, and human factors associated with repairs to concrete port and harbor structures.

It is recommended that future study or research in the field of repairs to concrete port and harbor structures be structured in a twofold manner. First, additional technical study to further quantify adhesion and durability of various repair materials under various environmental conditions will be useful to broaden the body of information available to engineers in the selection of a repair system for any given application.

Second, any technical or technological research should be coupled with application in actual practice. Where funds and opportunity permit, a hands on approach to independent study will serve to temper in practice that which is generated in the library and laboratory.

SECTION IX

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APPENDIX 1

WAVE ACTION CONSIDERATIONS FOR REINFORCED CONCRETE PORT AND HARBOR STRUCTURES

Introduction

In a properly designed harbor, wave forces are not typically of a magnitude which will directly govern the design of structures within the harbor. In most cases, the design of reinforced concrete piers, wharves, and associated structures is driven by operational loads that the structure will experience, namely: mooring loads, collision loads, and cargo handling live and dead loadsIt is important, however, to understand the effect of waves in a harbor on the deterioration of harbor structures, particularly the wave's ability to induce what is commonly known as "mechanical" damage. A thorough knowledge of the wave environment in a given harbor will allow for more accurate evaluation and effective correction of deterioration of the harbor structures. This summary is focused on the deterioration of reinforced concrete harbor structures, specifically the effect of wave action in causing and accelerating that deterioration.

I. Characterization of Wave Heights Within Harbors

Harbors effectively protected by breakwaters admit waves only through the harbor entrances which are required for ship traffic. A typical example of wave diffraction occurs in such a harbor. As waves enter through the harbor entrance, they expand throughout the entire harbor area in wave fronts which have their center at the harbor entrance. Their expansion is in ever broader circular wave fronts, and thus the energy per unit length in these wave fronts decreases with the distance away from the harbor entrance. The following equation by Stevenson may be used to approximate wave heights within a harbor. (Model tests may also be used for more detailed analysis of actual harbor design)

Hp = H [SQRT(b/B) -.02(1/4 th Root D) *(1+SQRT(b/B)]

Where:

Hp= height of reduced wave at any point, p, in harbor, ft.

H = height of wave at entrance, ft.

b = breadth of entrance, ft.

B = breadth of harbor at p, ft. (= length of arc with radius D and center at middle of entrance

D = distance from entrance to p, ft.

The equation does not apply to points less than 50 ft from the harbor entrance because the closed form above is based on the approximation of the harbor entrance being a point source for the incoming wave, when in fact the size (breadth) of the source is dependent on the breadth of the harbor entrance.

Another method for estimating wave heights in the lee of a breakwater is to utilize isoline plots of diffraction constants for various angles of wave incidence (after Wiegel, 1962). Similar to Stevenson's equation, this method calculates wave heights as follows:

$$H = K'Hi$$

Where: H = Local (diffracted wave height)

K' = diffraction coefficient (from isoline plot)

Hi = incident wave height

When using this method, the effects of refraction and shoaling must be considered, as the isoline plots are based on an assumption of constant depth. A sketch of wave diffraction is shown in Figure (1), followed by a typical wave diffraction diagram in Figure (2).

Typical harbor design calls for 4-foot limiting wave height for large vessels and non-resonance storm conditions. In the case of smaller craft, the limiting wave height should be on the order of 2 feet.

II. Natural Deterioration of Structures

Damage in its various forms affects structural capacity by (1) reducing or altering the net section geometry, (2) reducing the material strength, and (3) excessively deforming the member so that the applied load pattern is changed. In the case of wave forces on reinforced concrete harbor structures, the third factor, excessive deformation, is generally only a problem when related to the extreme mooring loads generated during extreme storm or seiche conditions, and not generally a result of pure wave loadings. (Seiche and storm waves will be addressed in detail in sections III and IV.) The first two factors, reducing net section geometry, and reducing material strength, are common symptoms of wave induced deterioration in harbor structures.

In the first case, reduction of net section geometry, wave action (and the related action of tidal and other currents) is directly responsible for one of two major types of structural abrasion in the "surf" or, in the case of harbor structures, "splash" zone.

Abrasion of concrete piles in the splash zone causes a reduction of the cross sectional area of the pile and the resultant increase in exposure of the reinforcing steel to other corrosive and abrasive actions. The specific mechanisms for the abrasive action include solid particles (sediments) and gaseous bubbles (entrained air, etc.) accentuating the erosion due to the flow of water around the pile. Figure (3) illustrates a typical concrete pile exposed to abrasion.

The second source of damage, reducing material strength, is most commonly a result of corrosion in reinforced marine concrete. That corrosion is most active in the splash zone which is in turn defined by both the range of the tide and by the amplitude of wave action in the harbor.

III. Storm Wave Damage

Storms that can generate winds of 60 knots and above are known as hurricanes in the Atlantic, Typhoons in the Pacific, and monsoons in the Atlantic Ocean. These have caused more damage in the United States over the years than any other type of natural disaster. Winds, heavy rains, and high tides which comprise the storm can cause major wave induced damage and loss of life. For example, In 1900, hurricane winds of some 120 knots struck the gulf area in the vicinity of Galveston, Texas, raising the normal expected tidal range

of 1.96 feet by an additional 15 feet or more. On top of that tidal surge, there were 23 foot wind waves which caused considerable damage.

This type of generated force can cause severe damage to an operational waterfront facility. Potential damage includes: moorings may be torn by vessels responding to storm induced wave motions as they move freely against fixed facilities and other moored vessels. In addition to the destruction of mooring fixtures, actual structural damage to reinforced concrete piers and wharves may result from storm induced impacts of moored vessels and cargo handling facilities. In some harbors, fendering systems include reinforced concrete piles designed to absorb mooring loads (fender piles). However, these piles are often subjected to damage from sloppy shiphandling or storm induced conditions. The calculation of these forces is addressed in section V. Regardless of the source of damage, fender piles must be repaired to insure that they retain fendering capacity for the next incident, be it storm or wayward conning officer.

IV. Seiche (Harbor Surge)

Another source of wave induced damage to reinforced concrete port and harbor structures is the influence of seiche motion on moored

ships. In simple terms, seiche is the long period agitation of harbor basins caused by the influx of long period ocean waves. The formal definition of seiche is "a standing wave oscillation of an enclosed body of water that continues, pendulum fashion, after the cessation of the originating force, which may have been seismic or atmospheric". Seiche is a phenomena associated with ocean waves having periods in excess of those of normal sea swell. Such waves, commonly known as long waves, have periods ranging from 20 seconds to several hours. Long waves exhibit relatively low heights, on the order of .1 to .4 feet. They are highly reflective, even off flat slope beaches, and will pass virtually unimpeded through porous breakwaters. Seiche occurs within a basin, harbor, or bay during certain critical wave periods when the period of the incident, long wave energy matches the resonating period of the basin. The result is a standing wave system comprising reinforced wave heights greater than those of the incident wave. The water surface exhibits a series of nodes and antinodes with respect to the water column. The range of the natural periods of harbor basins can be roughly the same as the natural period of motion of large moored ships. In this case the resulting resonance can increasingly agitate ships in their moorings and cause severe damage to ships, moorings, and reinforced concrete fendering and wharf systems. Figure (5) shows a typical one dimensional standing wave system and figure shows basin seiche potential for a sample basin.

Due to the nature of long waves, little can be done to prevent them from entering a harbor. There are, however, three basic approaches to mitigation of problems incurred by seiche:

1) Adjust the geometry or depth of the basin so that the natural oscillating period of the basin is adjusted away from the critical periods of the incident wave energy.

2) Locate berths within the harbor basin away from the seiche nodes in order to avoid regions where vessels are subject to large surging motions.

3) Tighten the mooring lines on vessels to aid in reducing the amplitude of surge movement when the vessel is subject to seiche. This also effectively increases the stiffness of the mooring, and reduces its response period, effectively away from the period of the seiche agitation.

V. Prediction of Forces Due to Storms and Seiche

Extreme ship motions, resulting from storm waves or seiche conditions at reinforced concrete piers, may cause a wide variety of damage to both ships and their piers. Damage may affect fender

systems, pier decks, pier structural piles, as well as the ships themselves and their mooring lines and fittings.

In an effort to quantify potential loads induced on pier structures and fender systems during seiche or storm conditions, it is helpful to utilize the docking forces analysis that is used during initial design of the pier. Docking impact design usually assumes that the maximum impact to be considered is that produced by a ship fully loaded (displacement tonnage) striking the dock at a shallow angle (say 10 degrees) to the face of the dock, with a velocity normal to the dock of .25 to .5 feet per second. This equates to a forward velocity of .8 to 1.75 knots, and fender systems are typically designed to absorb the energy of this docking impact. Where storm or seiche conditions cause velocities normal to the pier of greater than .5 feet per second, the energy of impact increases with the square of the velocity as in:

$$E = 1/2\ M\ v^2$$

If we assume that the point of contact of the bow with the fender is at the one fourth point of the length of the ship, then the energy to be absorbed by the fender system and the pier is assumed to be 1/2 E, as the remaining 1/2 is absorbed by the ship

and the water as the ship rotates around the point of contact. (After Quinn, 1972)

When vessels already moored are driven against the pier by storm or seiche action, more of the energy of impact is absorbed by the pier. The additional amount is dependent upon the form of the ship as it relates to the form of the pier. (Worst case is a flat ship in normal impact with a flat pier). This concept is seen graphically in Figure (6) which depicts an "eccentricity coefficient" (Ce) that may be used to factor the total impact energy for various ship forms against a flat pier.

To account for the shape of the ship as it relates to the shape of the pier, a geometric coefficient (Cg) is used. Geometric coefficients are highest for the unlikely case of a concave sided ship, and lowest for an extremely rounded convex ship.

The flexibility of the fender system is factored into the equation with a deformation coefficient (Cd), with a value of 1.0 for a flexible fender, decreasing with the flexibility of the fender.

The configuration of the pier is also a factor in the calculation of docking or collision energy, and a configuration coefficient (Cc) is applied to account for the porosity of the pier. An open (pile

supported) pier will have a configuration coefficient of 1.0, while a solid walled pier will have a configuration coefficient of .8 to account for the cushioning effect from the face of the pier.

An additional consideration in the calculation of the impact energy is the added mass of water moving along with the ship. This added mass may be approximated as a cylindrical column of water having the length of the vessel and a diameter equal to the draft of the vessel, and may be expressed as:

$$pi/4 * D^2 * L * \text{density of seawater}$$

where: D = draft of the ship

L = length of the ship

density of seawater = .0287 long ton/ cu ft

Rather than calculate separately the actual added mass of seawater, it is also common to use a mass coefficient (Cm) to modify the mass of the ship, typically Cm varies between 1.5 and 2.0, depending on the form of the ship, berthing (or surging) velocity, water depth and acceleration and deceleration of the ship.

In summary, collision or fendering loads as a result of storm or seiche conditions may be calculated using docking force analysis:

$$\text{Energy (fender)} = C_e\ C_g\ C_d\ C_c\ C_m\ 1/2\ MV^2$$

Where the coefficients are as described above. (after NAVFAC DM26.2)

Actual local loads are dependent upon the type and construction of the fender system. For reinforced concrete fender piles and relatively flat ship impacts resulting from storm or seiche action, loads may be assumed to be distributed equally over the number of piles impacted.

VI. Conclusion

As stated previously, a well designed harbor will not experience severe storm and seiche conditions as described above, and will have design features to mitigate the natural deterioration of reinforced concrete structures associated with wave action. However, because our analysis is centered on repair of deteriorated reinforced concrete harbor structures, we must examine means to mitigate the causes of damage so that repairs effected will have a longer useful life.

Examples of these types of mitigation include, augmented fender systems, wrapping or coating of reinforced concrete piles, selectivity in the types of ships moored at "high risk" berths, and reduction of pier appurtenances to reduce applied loads from wave action. Specific repair cases will require individual analysis, but by understanding the fundamental causes of mechanical damage, it is hoped that repairs will be engineered for longer life.

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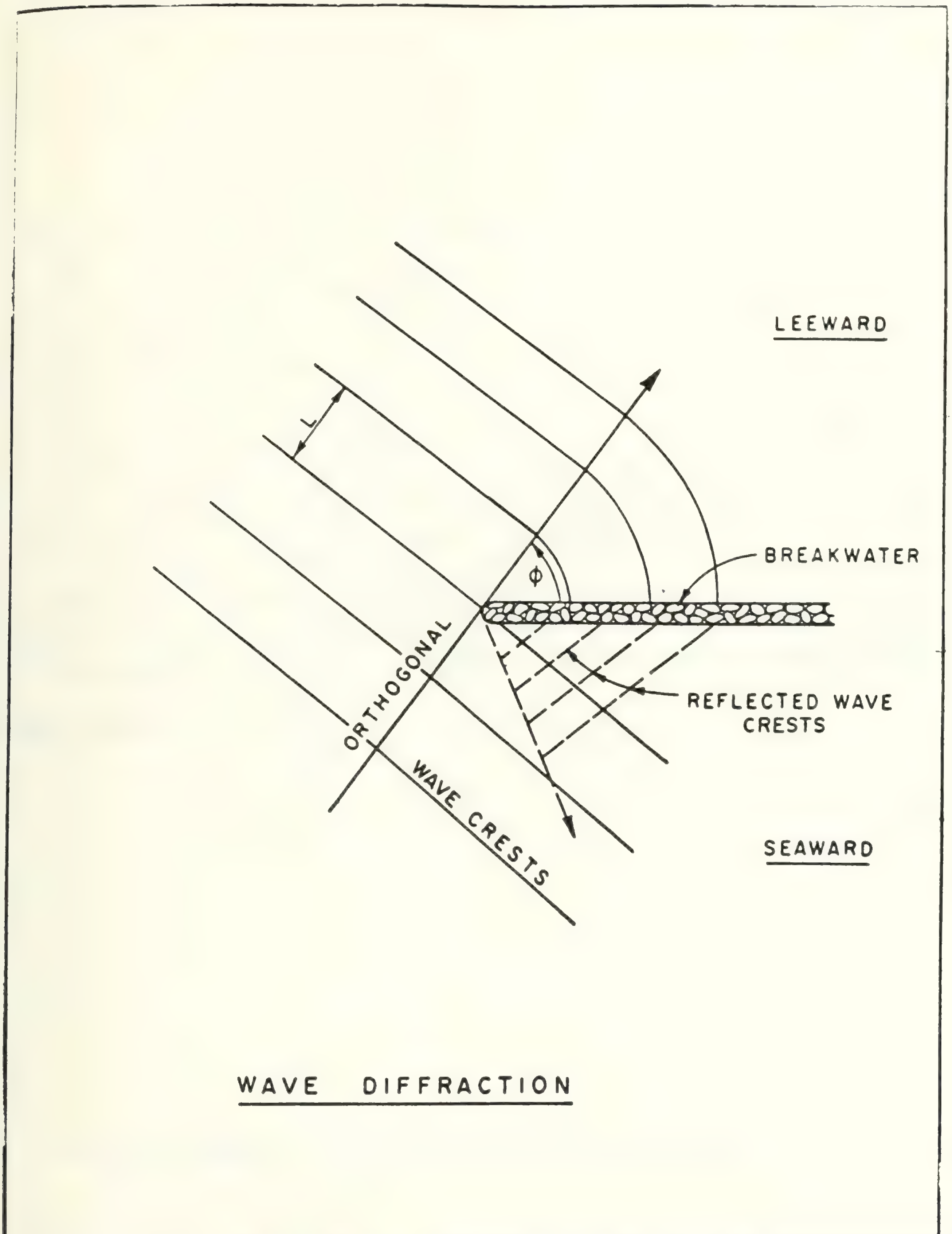

FIGURE (1) Wave Diffraction Around a Breakwater [After Wiegel]

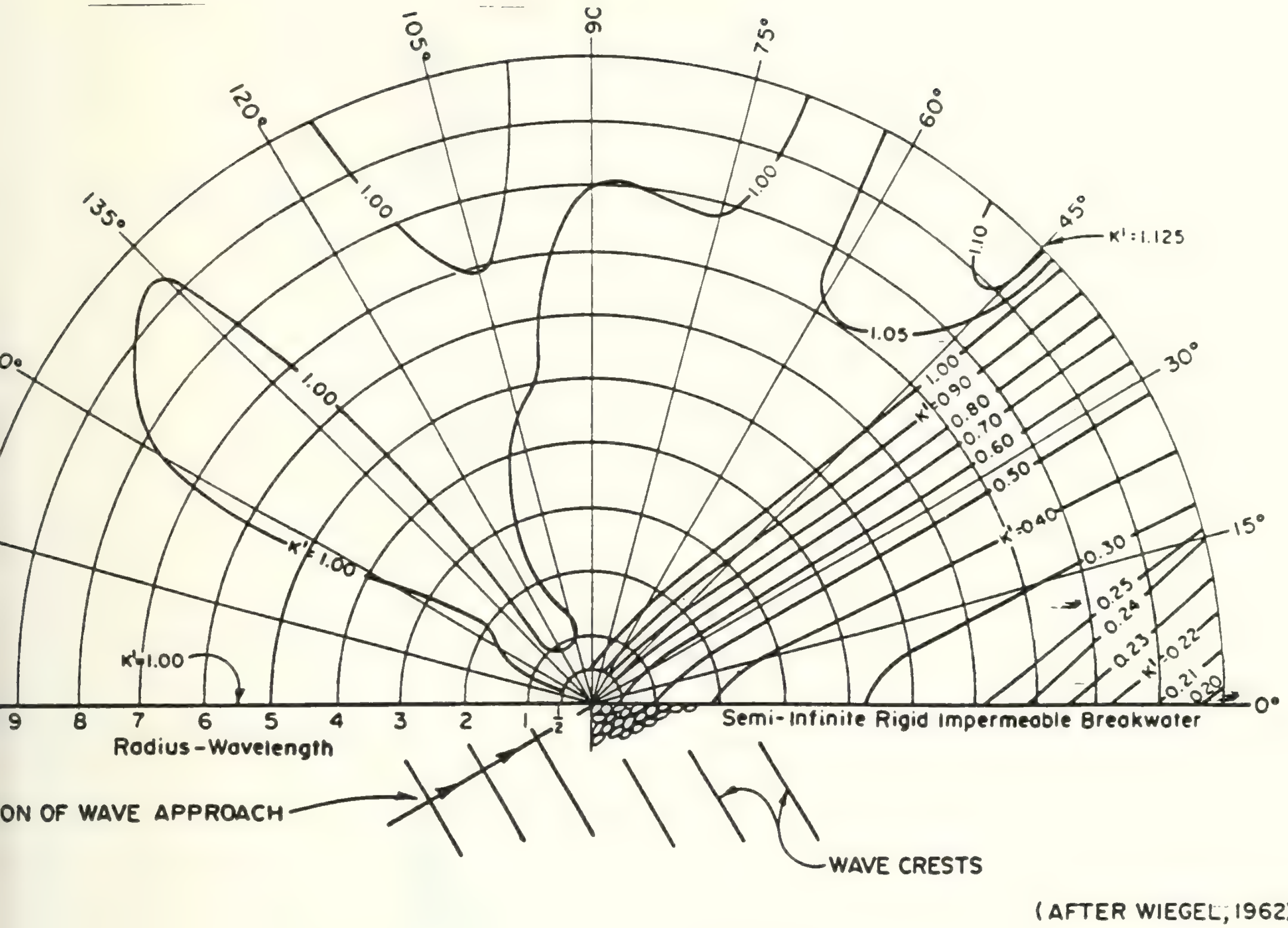

FIGURE (2) Representative Wave Diffraction Diagram [after Wiegel]

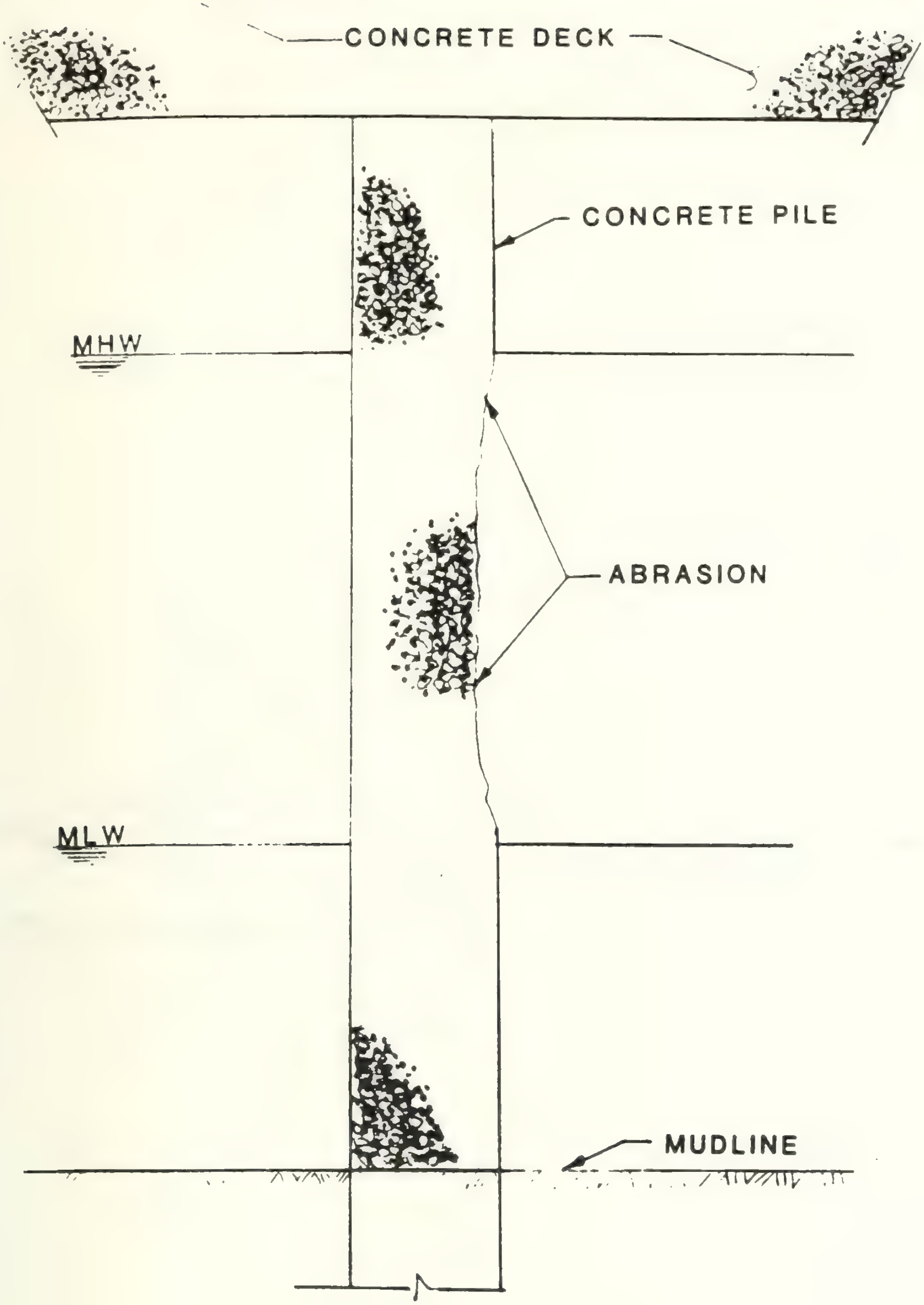

FIGURE (3) Abraded Concrete Pile [After NCEL]

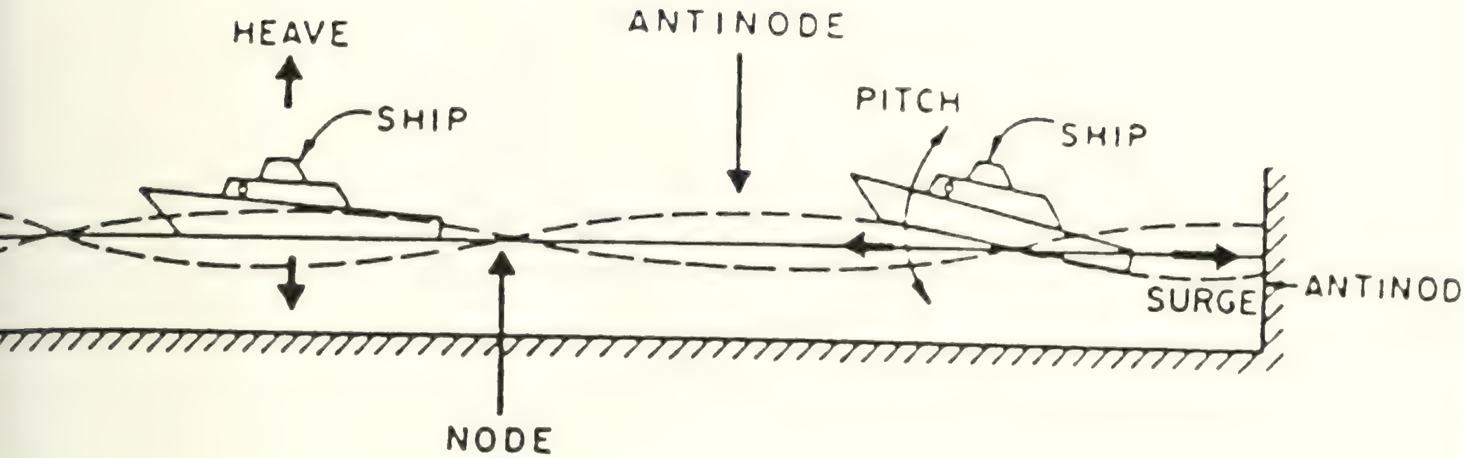

FIGURE (4) Typical standing Wave System in A Harbor

[After DM 26.2]

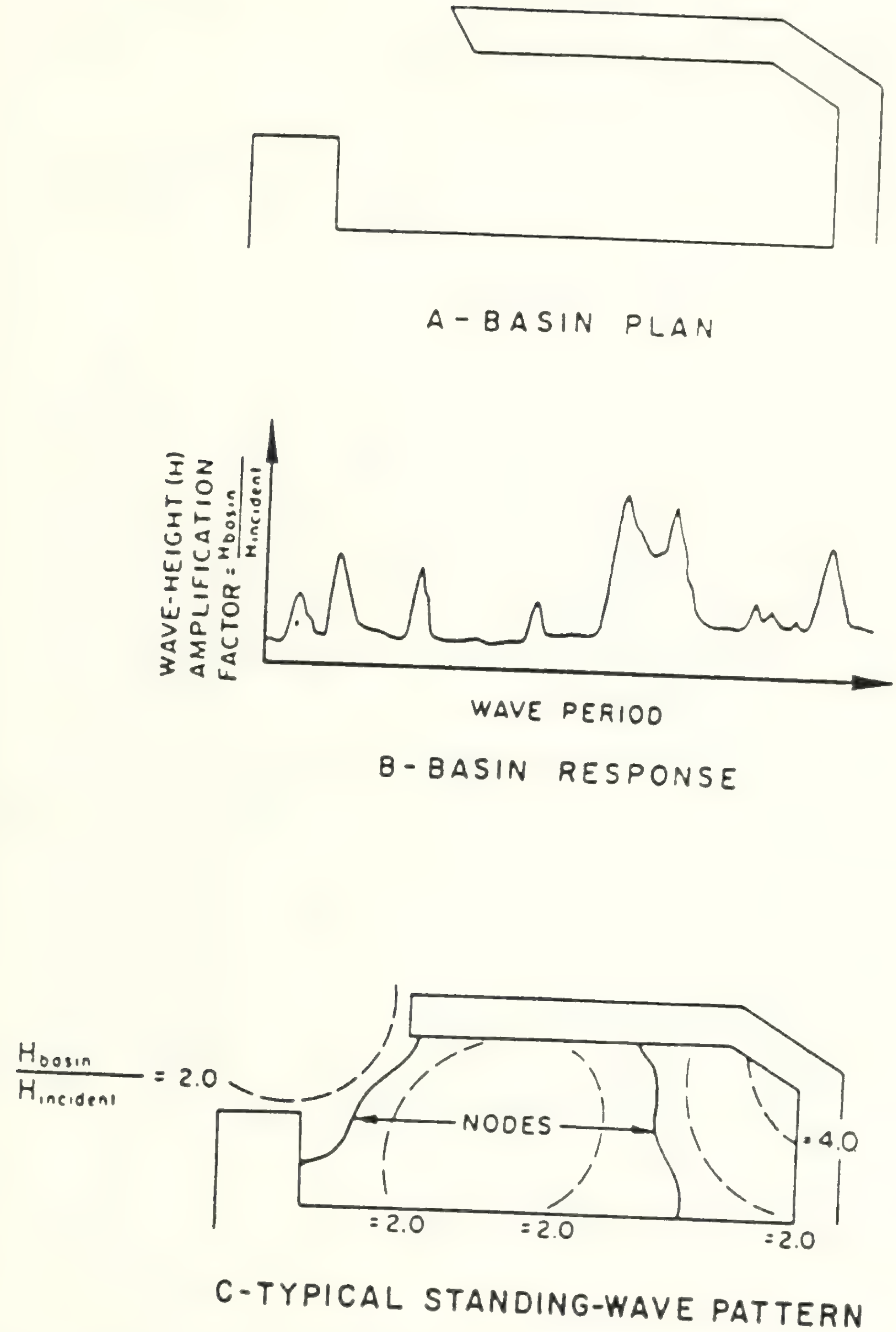

FIGURE (5) Basin Seiche Potential [After DM25.1]

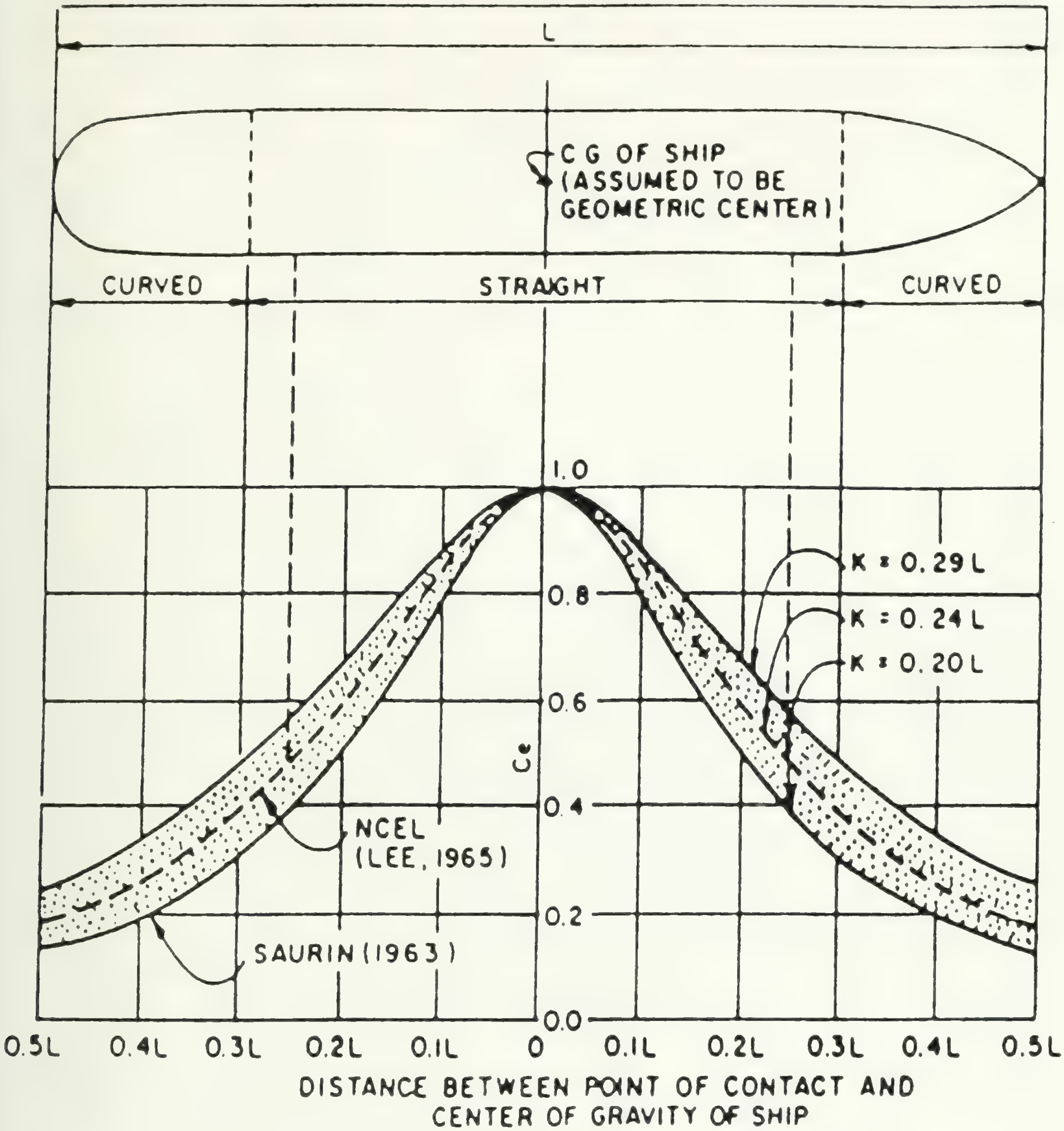

FIGURE (6) Eccentricity Coefficient For Calculation of Docking Force [after DM 25.1]

APPENDIX 2

LABORATORY AND FIELD EVALUATION

OF

FREEZING AND THAWING TESTS

LABORATORY AND FIELD EVALUATION OF FREEZING AND THAWING TESTS

by

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ABSTRACT

This paper traces the development of accelerated freezing and thawing tests in the U.S.A., and outlines the various freezing and thawing tests currently being used in North America and elsewhere. The outdoor exposure sites in the U.S.A., Japan and China for evaluating the freezing and thawing resistance of concrete are described, together with the nature of investigations being performed at these natural weathering sites. Special emphasis is given to the investigations being performed by CANMET and the Corps of Engineers of the U.S.A. Parameters to evaluate the freezing and thawing tests both in the laboratory and in the field are discussed. It is emphasized that the results obtained from the laboratory freezing and thawing tests cannot be correlated with those obtained from the tests at the natural weathering sites because the nature of the exposure in the two types of tests is essentially different.

INTRODUCTION

Concrete is one of the most versatile construction materials, and has a long documented history of excellent performance. Portland cement concrete with or without supplementary cementing materials need not deteriorate provided it is proportioned, made, placed, compacted and cured correctly. Performance of concrete under severe freezing and thawing conditions is no exception, provided concrete is air-entrained to provide adequate air-void parameters. The accelerated tests in a laboratory have become an accepted method to evaluate the freezing and thawing resistance of concrete. Also, the exposure of large concrete test specimens at natural weathering sites is slowly gaining acceptance as an alternate means to evaluate freezing and thawing of concrete, though this form of testing is slow and is of long-term nature. This paper traces the development of accelerated freezing and thawing tests in the U.S.A., and discusses the accelerated tests which are in current use in the U.S.A., Canada and Europe. CANMET investigations on the subject, and the current research studies being performed at natural weathering sites in the U.S.A., Japan and China are discussed. The difficulty of correlating the results of laboratory tests with those obtained from natural weathering sites is mentioned.

LABORATORY ACCELERATED FREEZING AND THAWING TESTS

Developments in the U.S.A.

In the late 1920's and early 1930's, extensive investigations were performed in the U.S.A. to delineate the parameters affecting the long-term durability of concrete. These studies led to a report entitled, "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete", and was published by the American Concrete Institute in 1940 (1). Many recommendations of this report including the maximum water-to-cement ratios for different exposure conditions are as valid today as they were in the 1940's. While the above research was being performed in the general area of durability, Scholer (2) published his laboratory results on the accelerated freezing and thawing of concrete. The publication of Scholer's, results combined with the availability of commercial freezing machines, gave considerable impetus to the use of accelerated freezing and thawing tests. Various leading U.S. agencies such as the Corps of Engineers, Bureau of Reclamation, Bureau of Public Roads and Portland Cement Association made specialized equipment for testing concrete by freezing and thawing (3).

In 1951, ASTM Committee C 09 on concrete and concrete aggregates formed a sub-committee with the main task of developing test methods for evaluating the resistance to freezing and thawing of aggregates in concrete. In 1953, four methods were standardized by the ASTM. These were:

- (i) Resistance of Concrete Specimens to Rapid Freezing and Thawing in Water;
- (ii) Resistance of Concrete Specimens to Rapid Freezing in Air and Thawing in Water;

(iii) Resistance of Concrete Specimens to Slow Freezing and Thawing in Water;

(iv) Resistance of Concrete Specimens to Slow Freezing in Air and Thawing in Water.

ASTM Test Method C 666

In 1971, the slow test methods were withdrawn by ASTM, and the two rapid methods were combined into one test method with two test procedures. The latter was designated as ASTM Test Method C 666, and is currently the most commonly used test method in North America. Briefly, the test method consists of alternatively lowering the temperature of specimens from 4.4 to -17.8°C and raising it from -17.8 to 4.4°C in not less than 2 h or more than 5 h. Thus a minimum of 5 and a maximum of 12 complete cycles can be achieved during a 24 h period. The testing is commenced after concrete specimens have been moist-cured for 14 days, and is deemed completed after 300 cycles (4). The scope and the significance and use statement of the test method are as follows.

Scope

This test method covers the determination of the resistance of concrete specimens to rapidly repeated cycles of freezing and thawing in the laboratory by two different procedures: Procedure A, Rapid Freezing and Thawing in Water, and Procedure B, Rapid Freezing in Air and Thawing in Water. Both procedures are intended for use in determining the effects of variations in the properties of concrete on the resistance of the concrete to the freezing-and-thawing cycles specified in the particular procedure. Neither procedure is intended to provide a quantitative measure of the length of service that may be expected from a specific type of concrete.

Significance and Use

As noted in the scope, the two procedures described in this test method are intended to determine the effects of variations in both properties and conditioning of concrete on the resistance to freezing and thawing cycles specified in the particular procedure. Specific applications include specified use in Specification C 494, Method C 233, and ranking of coarse aggregates as to their effect on concrete freeze-thaw durability, especially where soundness of the aggregate is questionable.

It is assumed that the procedure will have no significantly damaging effects on frost-resistant concrete which may be defined as (1) any concrete not critically saturated with water (that is, not sufficiently saturated to be damaged by freezing), and (2) concrete made with frost-resistant aggregates and having an adequate air-void system that has achieved appropriate maturity and thus will prevent critical saturation by water under common conditions.

If as a result of performance tests as described in this test method, concrete is found to be relatively unaffected, it can be assumed that it was either not critically saturated, or was made with "sound" aggregates, a proper air-void system, and allowed to mature properly.

No relationship has been established between the resistance to cycles of freezing and thawing of specimens cut from hardened concrete and specimens prepared in the laboratory.

Evaluation of Test Results of ASTM Test Method C 666

The deterioration of test specimens in ASTM Test Method C 666 is determined by the resonant frequency method. The fundamental transverse frequency is determined at frequent intervals and is used to calculate the relative dynamic modulus of elasticity P_c as follows:

$$P_c = \frac{n_1^2}{n^2} \times 100$$

where P_c (in per cent) = relative dynamic modulus of elasticity, after c cycles freezing and thawing

n = fundamental transverse frequency at 0 cycles of freezing and thawing, and

n_1 = fundamental transverse frequency after c cycles of freezing and thawing.

The durability factor is calculated as:

$$DF = PN/M$$

where DF = durability factor of the specimens

P = relative dynamic modulus of elasticity at N cycles, per cent

N = number of cycles at which P reaches the specified minimum value for discontinuing the test or the specified number of cycles at which the exposure is to be terminated, whichever is less, and

M = specified number of cycles at which the exposure is to be terminated.

Shortcomings of ASTM Test Method C 666

Notwithstanding the fact that ASTM Test Method C 666 has gained wide acceptance in North America and elsewhere, it has several serious limitations. These are:

(i) The freezing rates (up to 12°C/h) are unrealistically high as compared with natural cooling rates that seldom exceed 3°C/h.

(ii) Generally, the specimens are subjected to the accelerated test after 14 days of moist-curing. This does not allow any drying of the specimens as opposed to natural field exposure where some drying always takes place.

(iii) Variability of the results both within and between laboratories is high.

(iv) Primary measure of deterioration is the relative dynamic modulus calculated from fundamental transverse frequency determinations; length change is an optional requirement and is not given equal consideration with the relative dynamic modulus.

(v) The degree of restraint offered by the containers in Procedure A can influence the number of cycles required to reach a specified level of relative dynamic modulus.

(vi) The commercially available equipment frequently breaks down and the containers in Procedure A have to be frequently replaced.

(vii) There is a lack of uniformity in the temperature of the cabinet of the most commonly used commercially available freezing and thawing equipment.

Dilation Method (ASTM Test Method C 671)

To overcome some of the shortcomings of ASTM Test Method C 666, accelerated method of freezing and thawing. In 1955, Powers (5) proposed a critical dilation method. In 1961, the California Division of Highways adopted the Powers' proposed method for evaluating aggregates for highway construction (6). Larson and his associates (7) also evaluated the above test method and suggested several modifications. In 1971, ASTM standardized the above test as ASTM Test Method C 671 (8). Briefly, this method provides for the fabrication of 75 by 150-mm cylindrical specimens that are stored in water at 1.7 ± 1.1°C. At two week intervals, the specimens are cooled in water-saturated kerosene from 1.7 to -9.4°C at a slow cooling rate of 2.8 ± 0.5°C/h. During the slow cooling cycle, the specimen is kept in a strain frame to allow measurement of length change. A typical plot of length change versus temperature is shown in Fig. 1. Prior to critical saturation the length will proceed along the dashed curve without dilation. Critical dilation is defined as a sharp increase (by a

factor of 2 or more) between dilations on successive cycles (Fig. 1). Highly frost resistant concrete may never exhibit critical dilation. The scope and the significance and use statement of the test method are as follows:

> <u>Scope</u>
>
> This test method covers the determination of the test period of frost immunity of concrete specimens as measured by the length of time of water immersion required to produce critical dilation when subjected to a prescribed slow-freezing procedure.
>
> <u>Significance and Use</u>
>
> This test method is suitable for ranking concretes according to their resistance to freezing and thawing for defined curing and conditioning procedures. The significance of the results in terms of potential field performance will depend upon the degree to which field conditions can be expected to correlate with those employed in the laboratory.
>
> This test method is also suitable for use by those people engaged in research and development in the field of concrete durability. It is believed that a fundamental understanding of length change of concrete subjected to freezing will lead to more durable concrete.

Although the dilation test is more elegant and the needed equipment is relatively inexpensive, the test method is not being used widely. Newlon (3) has tried to explain the reasons for this as follows:

> "ASTM Test C 671 and Recommended Practice C 682 have not been used extensively. Although the apparatus is comparatively inexpensive and larger numbers of specimens can be processed than with ASTM Test C 666 equipment, significant storage capacity is required. The procedures of ASTM Recommended Practice C 682 are extensive and complex, largely because of requirements designed to bracket a broad range of potential exposure conditions. It is suggested that aggregates and concrete be "maintained or brought to the moisture condition representative of that which might be expected in the field". However, it is noted that "aggregate moisture states other than dry or saturated are very difficult to maintain during preparation of specimens. Reproducibility of overall test results is likely to be affected adversely by variability in aggregate moisture". While the complexity of evaluation procedure limits its general applicability, where large projects or economic consequences of detailed aggregate evaluation are warranted, the procedures may be justified".

Developments in Europe

In addition to the research and development associated with the accelerated freezing and thawing tests in North America, Sweden has been in the forefront in Europe in investigations dealing with durability of concrete. Fagurlund of Sweden (9,10) has best described the current European practice as follows:

"RILEM Committee 4-CDC, "Durability of Concrete", has proposed three methods of carrying out freeze-thaw tests of concrete. One of them is a test where the ability of a concrete surface to sustain the combined effect of freeze-thaw and deicing chemicals is investigated. The method of test is described in detail elsewhere (11).

The second method is a more traditional freeze-thaw cycling test without chemicals. It differs, however, from normal tests in so far as the specimens are sealed during all 25 to 50 cycles, which means that the original water content, which corresponds to 28 days of water storage, is kept almost constant during the whole test.

The method of test is described in detail elsewhere (11).

The third test is, however, quite new in its character. It is called "The critical degree of saturation method of assessing freeze-thaw resistance of concrete". The method is based on the existence of critical moisture contents or degrees of saturation at freezing.

The critical degree of saturation, S_{CR}, is determined by a test in which sealed specimens containing different amounts of water are subjected to a few freeze-thaw cycles.

Other specimens are subjected to a test in which their ability to absorb water is measured. This test yields a sort of potential degree of saturation which can be reached during very moist conditions. It is called the capillary degree of saturation, S_{CAP},

The freeze-thaw resistance, F, is then defined;

$$F = S_{CR} - S_{CAP}$$

S_{CR} is supposed to be almost independent of outer climatic conditions. S_{CAP} on the other hand, increases with increasing time for water uptake. Theoretically, for each type of concrete, each environment could be translated to a certain water uptake time of the specimen and, hence, to a certain S_{CAP}. Therefore, F expresses a sort of potential freeze-thaw resistance of a given concrete type used in different environments".

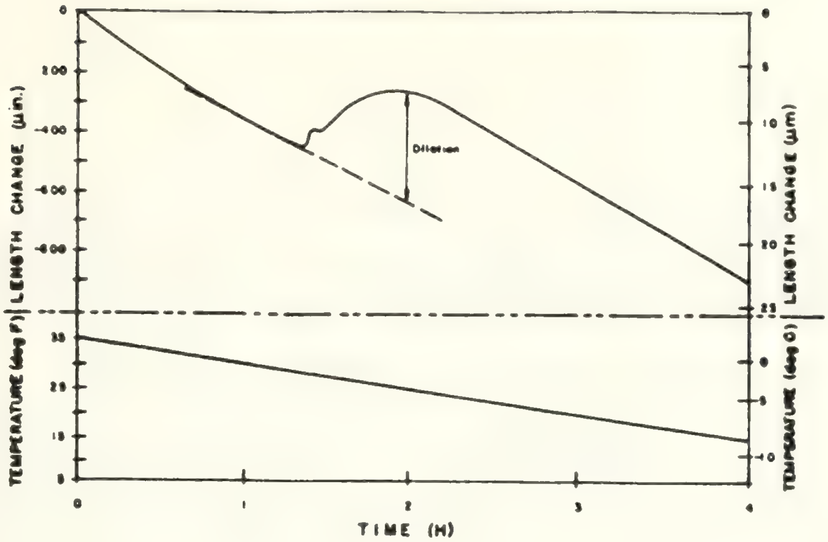

Fig. 1 - Typical Length Change and Temperature Charts for the Dilation Method.

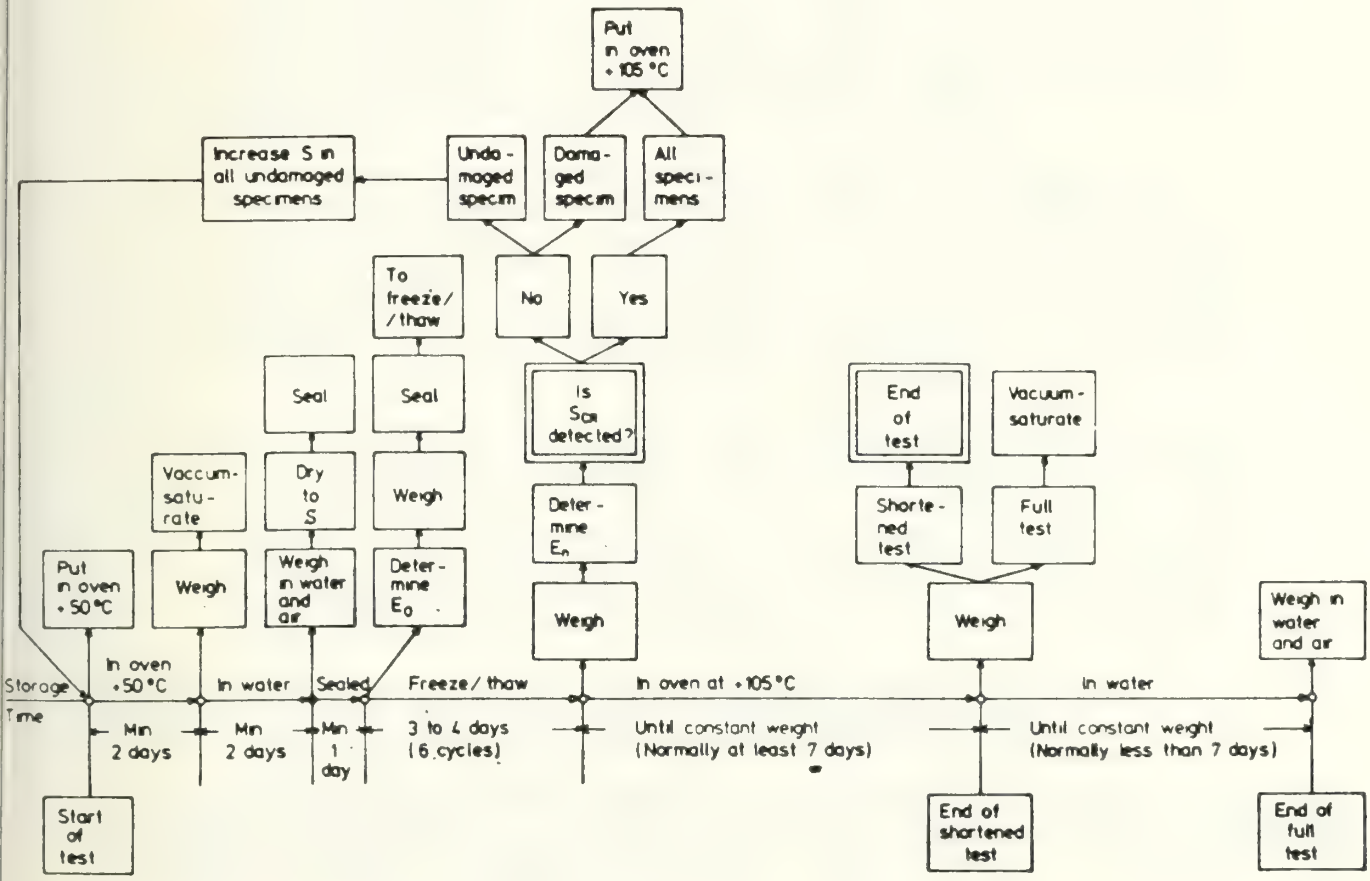

Fig. 2 - Graphical Representation of Critical Degree of Saturation Test Method.

A graphical representation of the critical degree of saturation test method is given in Fig. 2. In discussing the differences between the conventional accelerated freezing and thawing tests and the critical degree of saturation approach Fagurlund (10) goes on to state as follows:

> "There is a fundamental difference between the critical degree of saturation method and traditional methods; for example the second RILEM 4-CDC-method mentioned above. In the latter method, only one single degree of saturation is investigated. The test will only tell whether the concrete is damaged at the water content or not. In case of no damage, the test will tell nothing about the effect of even a rather small transgression of the water content tested.
>
> In such traditional tests where the specimens are stored in water for some periods, the water content, especially in the surface part, normally increases with increasing number of cycles and time of water storage. Hence, the same specimen will freeze with many different, undefined water contents. At some time-point, damage occurs since the water content at freezing was too high. The judgement of frost-resistance will, therefore, depend on the choice of the total number of cycles and the time of water storage during each cycle.
>
> Hence, by traditional methods, little information is obtained with regard to the safety against frost damage in the actual environment, In order not to overrate the frost-resistance, one, therefore, has to choose a test method which introduces somewhat more water in the material than can be expected in practice. The correct choice and control of water content in the test is a very intricate problem.
>
> By the critical degree of saturation method, the risk of frost damage can be estimated more easily since the freeze-thaw test is completely separated from the problem of finding out the moisture content appearing in practice. One can directly estimate the damage-risk caused by an increase in water content by looking upon the experimentally determined relation between frost-damage and degree of saturation".

CANMET Investigations on the Freezing and Thawing Resistance of Concrete Incorporating Superplasticizers and Silica Fume

During the past 25 years CANMET has performed extensive investigations on the freezing and thawing resistance of concrete using ASTM Test Method C 666, Procedure A, "Freezing and Thawing in Water" and Procedure B, "Freezing in Air and Thawing in Water" (12,13).

The concretes investigated included non-air-entrained, air-entrained, air-entrained and superplasticized, and silica fume concretes. Also investigated were high-strength lightweight concretes incorporating superplasticizers and silica fume (14). The conclusions from the results of some of these investigations are reproduced below:

Non-Air-Entrained Concrete (From Reference 12)

"Non-air-entrained concrete prisms, regardless of the water-to-cement ratio when tested in accordance with ASTM C 666 Procedure A (freezing in water and thawing in water) show very low durability factors (<31).

Non-air-entrained concrete prisms with water-to-cement ratios of 0.50 to 0.70 show very low durability factors (<12) when tested in accordance with ASTM C 666 Procedure B (freezing in air and thawing in water). However, concrete prisms with a water-to-cement ratio of 0.35 show satisfactory resistance to 300 cycles of freezing and thawing (durability factor >90). Between 300 and 400 cycles the prisms deteriorate markedly. Notwithstanding the strength loss associated with air entrainment, the use of air-entrained concrete is recommended for low water-to-cement ratio concrete for added assurance against damage when concrete is to be subjected to repeated cycles of freezing and thawing".

Air-Entrainend (From Reference 12)

"Regardless of the water-to-cement ratio used, the air-entrained concrete prisms show high durability factors (>90) at the end of 300 cycles of freezing and thawing. This is true for both ASTM C 666 Procedures A and B. However, beyond 300 cycles of freezing and thawing, concrete prisms with a water-to-cement ratio of 0.70 seem to perform poorly in test Procedure A as compared with test Procedure B".

Air-Entrained Superplasticized Concrete (From Reference 12)

"The bubble spacing factors $\bar{L}$ for air-entrained superplasticized concrete are, in some cases, higher than 0.20 mm. But in spite of the higher values of $\bar{L}$, and regardless of the test procedure used, the durability factors for these concretes are high (>90) at the end of 300 cycles of freezing and thawing for concrete mixes of Series I, and at the end of more than 700 cycles of freezing and thawing for concrete mixes of Series II. These durability factors are comparable with those for air-entrained concrete".

Silica Fume Concretes (From Reference 13)

"Non-air-entrained concrete prisms, regardless of the W/C+S and irrespective of the silica fume content used, show very low durability factors when tested in accordance with ASTM C 666, Procedure A. Therefore the use of non-air-entrained concrete is not recommended when it is to be subjected to repeated cycles of freezing in water and thawing in water. Whether non-air-entrained concrete will perform satisfactorily if it has greater maturity or the opportunity to dry before being subjected to the freezing and thawing tests is open to question. Only additional research can answer this question.

Air-entrained concrete with W/C+S of 0.35 and 0.30 and without silica fume performs satisfactorily when tested in accordance with ASTM C 666, Procedure A. However, concrete prisms incorporating 10 and 20% silica fume show poor performance in spite of having had more than 4% air in fresh concrete. There is evidence of increasing distress in test prisms with increasing amounts of silica fume. The relatively poor performance of silica fume concrete may be attributed to the unsatisfactory $\bar{L}$ values in hardened concrete even with more than 4% entrained air in fresh concrete. Caution should therefore be exercised when using very high percentages of silica fume in concrete if these concretes are to be subjected to repeated cycles of freezing in water and thawing in water and are likely to attain saturation levels comparable to those reached in the laboratory tests. Under these conditions, every attempt should be made to obtain very low $\bar{L}$ values in hardened concrete".

High-Strength, Air-Entrained Lightweight Concrete

The semi-lightweight structural concretes with or without supplementary cementing materials performed excellently in the freezing and thawing tests made in accordance with ASTM C 666 Procedure A. The durability factors at the completion of 500 cycles of freezing and thawing were greater than 90 (14).

The type of conditioning of the specimens, i.e. 35-day moist curing followed by 17 days drying in the laboratory air and finally five days of moist curing or moist curing for 56 days, did not affect significantly the durability factors.

The semi-lightweight concretes were made using an expanded shale aggregate of Canadian origin. The compressive strength of 150 x 300-mm cylinders were of the order of 66 MPa at one year.

FREEZING AND THAWING EVALUATION OF CONCRETE SPECIMENS AT NATURAL WEATHERING EXPOSURE SITES

In order to overcome some of the limitations associated with laboratory accelerated freezing and thawing tests, several organizations have established natural weather exposure sites. Large concrete specimens are placed at these sites and these are monitored over long periods of time to determine the effect of natural freezing and thawing cycles, and/or marine environment. One of the well-known exposure sites is operated by the U.S. Corps of Engineers, in Maine, U.S.A. The other recent exposure sites are in Japan and China. Ontario Hydro in Canada had an outdoor exposure site in the Toronto area but it appears to be no longer operational. The older exposure sites at Ostend, Belgium, Whihelm-Shaven, West Germany and Trondheim, Norway are not covered here.

The locations of the natural weathering sites in the U.S.A., Japan and China are described below, together with the nature of long-term research being performed at these sites.

U.S. Corps of Engineers Natural Weathering Site at Treat Island, Maine

In 1936, the Corps of Engineers of the U.S.A. established an outdoor exposure facility at Treat Island, in Cobscook Bay near Eastport, Maine. This facility has been in use since then, and is an ideal location for exposure tests, providing twice-daily tide reversals, together with extremes of temperature in winter. The exposure facility has been described in great detail by Thornton (15) and others (16,17,18,19). Briefly, the exposure site presently consists of a wooden rack approximately 37 x 12 m and adjoining wharf approximately 18 x 10 m that supports a wooden equipment room, and a portion of beach approximately 61 m long used for exposure of large specimens. According to Thornton (15), the specimens are installed at mean-tide elevation. The alternate conditions of immersion of the specimens in sea water and exposure to cold air, results in the concrete specimens being exposed for a large number of freezing and thawing cycles. The tides range from a high of about 8.5 m to a low of about 4 m with a mean of about 5.5 m. A temperature measuring device is embedded in the centre of a concrete specimen, records these temperatures. A cycle of freezing-and-thawing consists of the reduction of the temperature at the centre of a concrete specimen to below -2°C, and its subsequent rise to above that value. In 26 winters, from 1953 to 1979 the number of annual cycles of freezing and thawing has ranged from 71 to 185 with the average being 133. In recent years, the winters have been less severe, and from 1979 to 1988, the number of annual cycles has ranged from 26 to 153 with the average being 91.

Investigations Sponsored by the Corps of Engineers and Other U.S. Agencies at Treat Island, Maine

According to Scanlon (16) test specimens from 36 active programs are exposed at Treat Island. The investigations include those dealing with performance of prestressed concrete, fibre concrete, polymer concrete, roller compacted concrete and superplasticizers concrete. The results of these investigations have been published by Scanlon (16), Mather (17), O'Neil (18) and Schupack (19).

Size of Specimens

Typical size of a mass concrete specimen is 457 x 457 x 914 mm (18 x 18 x 36 in.). This size and type of test specimen is more amenable to measurements of transverse resonant frequency and provides long path length for pulse velocity measurements.

Evaluation Criteria

According to Thornton (15) test specimens are regarded as having failed when:

a) they separate into pieces
b) when the % dynamic modulus of elasticity, E, reaches 50 or less, or
c) when deterioration has progressed to such a point that reliable determinations of fundamental resonant frequency and pulse velocity cannot be obtained.

CANMET Sponsored Investigations at Treat Island, Maine

In 1977, CANMET sponsored a research program for the determination of durability in marine environment of portland cement concrete incorporating blast-furnace slag, fly ash, silica fume and superplasticizers (20,21). A series of 175 concrete prisms, 305 x 305 x 915 mm in size were cast over the eleven year period (1978-1989). The prisms were positioned at mid-tide level on a rack at the exposure site of Treat Island*, Maine.

The test specimens are monitored at yearly intervals: the specimens are photographed and rated on a visual basis. Ultrasonic pulse velocity is also determined. A brief description of the various phases of the research program is as follows (21):

Phase I: Durability of air-entrained concrete incorporating pelletized blast-furnace slag, and air-entrained and non-air-entrained concrete incorporating superplasticizers.

Phase II: Durability of air-entrained concrete incorporating fly ash and pelletized blast-furnace slag.

Phase III: Durability of air-entrained semi-lightweight concrete incorporating pelletized blast-furnace slag.

Phase IV: Durability of air-entrained concrete incorporating fly ash.

Phase V: Durability of air-entrained concrete incorporating granulated blast-furnace slag, and air-entrained and non-air-entrained concretes incorporating silica fume.

*With approval and co-operation of the U.S. Corps of Engineers, Vicksburg, Mississippi, U.S.A.

Phase VI: Durability of air-entrained semi-lightweight concrete incorporating fly ash and silica fume (with and without steel fibres).

Phase VII: Durability of air-entrained semi-lightweight concrete incorporating silica fume (batched at a ready-mix plant).

Phase VIII: Durability of air-entrained superplasticized concrete incorporating high volume of fly ash.

Phase IX: Durability of air-entrained concrete incorporating fly ash or pelletized blast-furnace slag or silica fume (with steel reinforcement).

The following conclusions are based upon the exposure of the test prisms at Treat Island for a relatively limited period of time, i.e. up to a maximum of nine years. Some of these will have to be reconsidered after the test prisms have been at the exposure site for 20 to 25 years. Furthermore, these conclusions are applicable only to the brands and types of the materials used in the investigation, and may or may not apply to other types of materials or similar materials from other sources.

Phase I

The surface scaling of test prisms increases with an increase in the water-to-(cement + slag) ratio. At higher ratios, the scaling also increases with increasing percentages of the pelletized slag (Fig. 3).

The types of cement used do not affect the performance of concrete investigated.

Non-air-entrained concrete is not durable under the exposure conditions experienced at Treat Island.

Phase II

Concrete incorporating high volumes of pelletized slag and fly ash perform relatively poorly. It appears that, for the combination of the materials used in this phase and for satisfactory performance under the exposure conditions at Treat Island, the air-entrained concrete must contain a minimum amount of cement and this appears to be of the order of 200 to 250 kg/m^3 (Fig. 4).

There is no significant difference in the performance of concretes made with ASTM Types I, II and V cements.

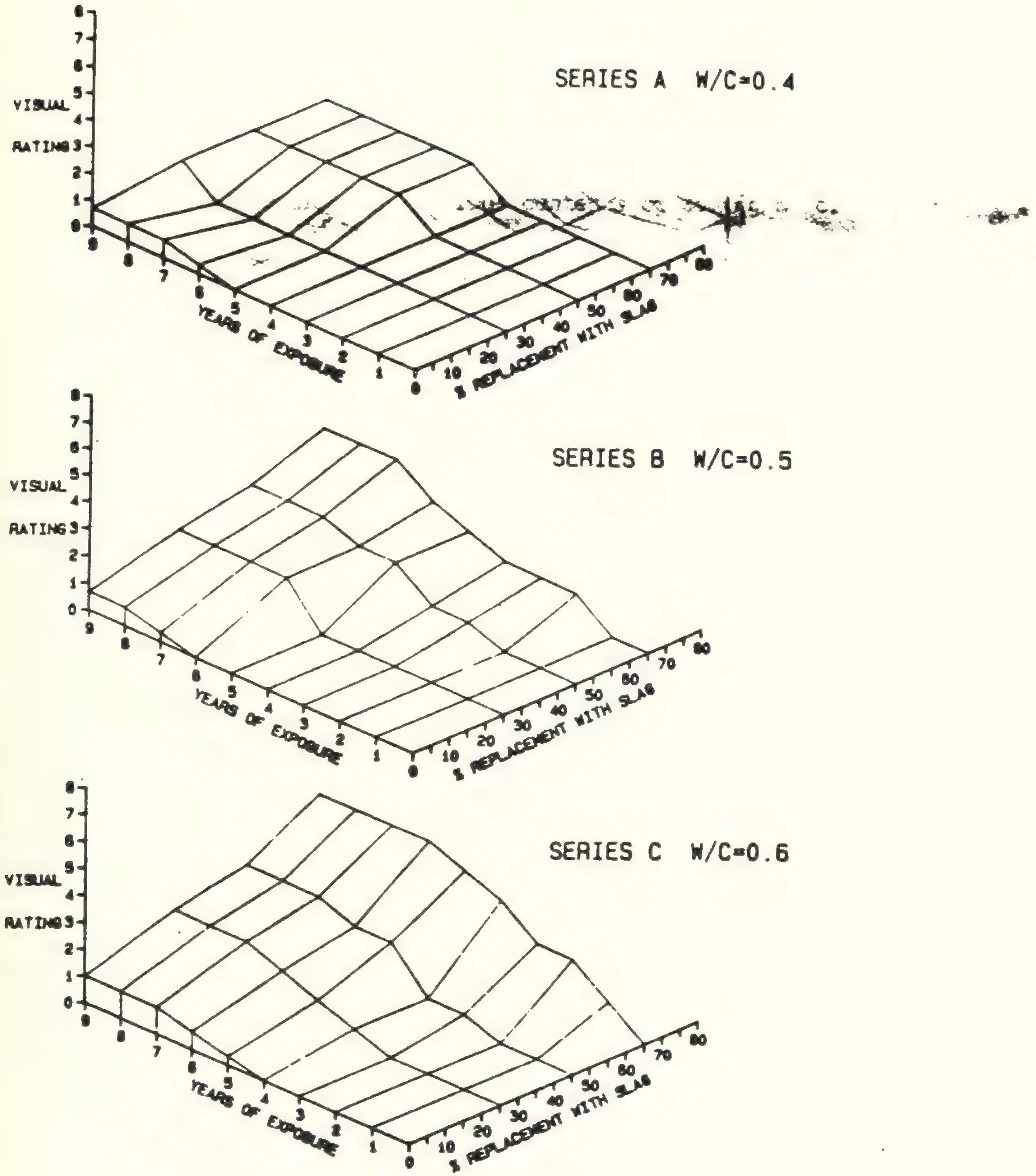

Fig. 3 - Visual Ratings as a Function of Both Slag Content and Time of Exposure (CANMET Investigations--Phase I).

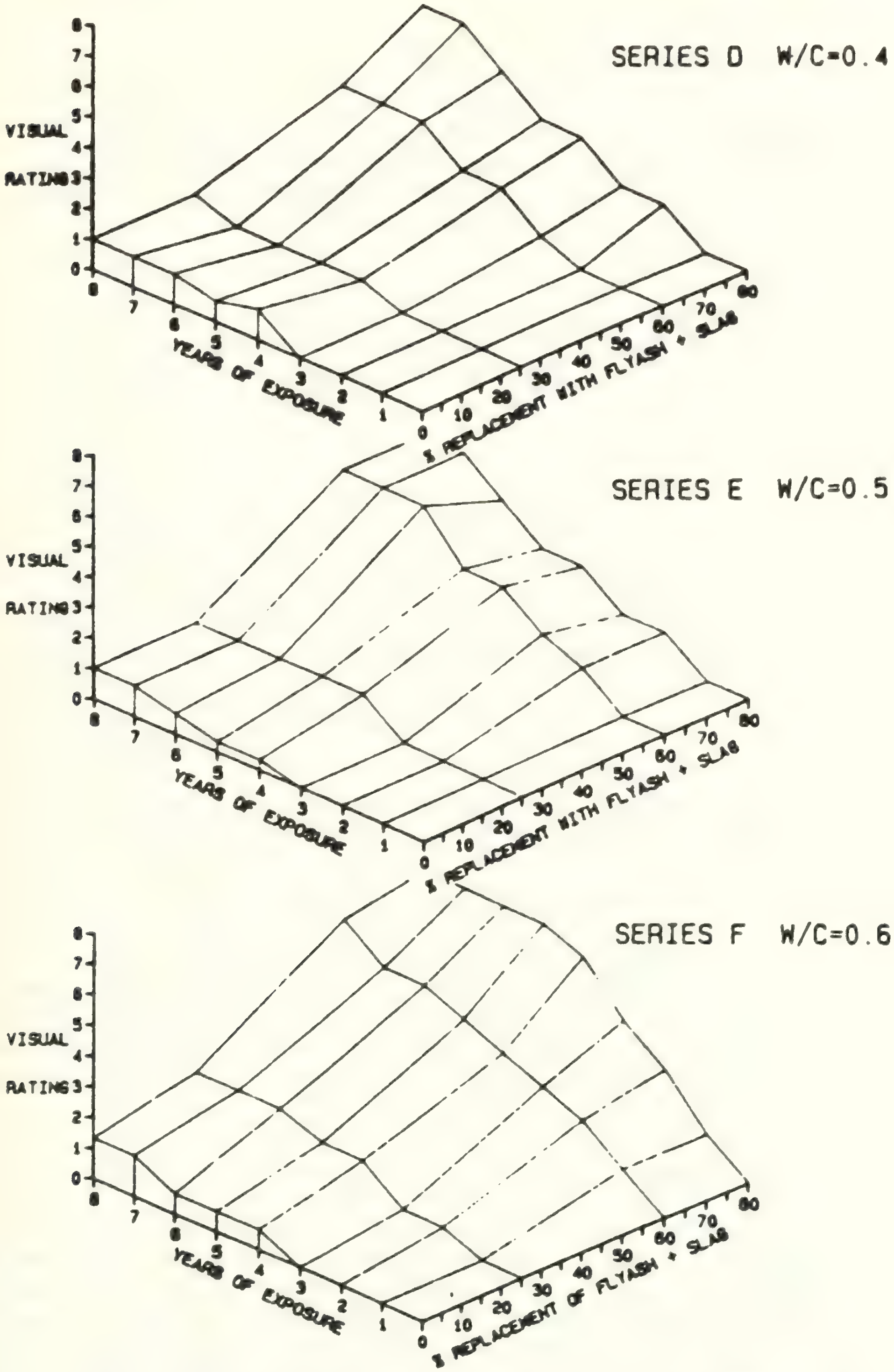

Fig. 4 - Visual Ratings as a Function of Both Fly Ash and Slag Content, and Time of Exposure (CANMET Investigations --Phase II).

Phase III

Semi-lightweight concrete made with expanded shale aggregate and having cementitious contents of 360 kg/m^3 or greater, perform satisfactorily.

Phase IV

Fly ash concrete at 25 per cent cement replacement level by mass can be satisfactorily used for the exposure conditions investigated, provided that the water-to-cementitious material ratio does not exceed 0.50.

Phase V

No firm conclusion can be drawn as to the performance of other concrete prisms which have been at the exposure site for a period of less than 5 years. This includes concrete prisms incorporating 80 per cent granulated slag as a replacement for cement and prisms incorporating silica fume.

Phases VI to IX

The very short exposure time for these phases do not allow the drawing of any conclusions at this stage.

Outdoor Exposure Sites at Ishikari and Bibi, Japan

The outdoor exposure sites where test specimens are being subjected to freezing and thawing cycling are located at Ishikari and Bibi, on the north Island of Japan. According to Shimada et al (22), the above sites for exposure sites were selected by taking into consideration the number of freezing and thawing cycles per year, access and availability of electricity.

The average number of freezing and thawing cycles per year at the above sites is about 50 with specimens freezing at -5°C and thawing at 0°C.

At each exposure site, in addition to the measurement of atmospheric temperatures, measurements of temperatures of the exposed specimens are also made by thermocouples which are embedded in specimens at a depth of 1 cm. In fine weather there is a difference of 7 to 8°C depending on the orientation of the specimens i.e. whether they were facing towards North or South; in cloudy weather the difference is only about 2°C.

The concrete test specimens were installed at the Bibi exposure site in 1971 and at the Ishikari exposure site in 1980, with the following objectives:

(a) to clarify the effect of various factors such as mixture proportions of concrete on freezing and thawing resistance of concrete.

(b) to establish correlations, if any, between the results of accelerated freezing and thawing performed in accordance with ASTM Test Method C 666 and the results obtained from the exposure under the natural environmental conditions.

Size of Specimens Used

The specimens used at the above sites were smaller than those which are being used at Treat Island. Generally the specimens were 150 x 300 mm cylinders or prisms of the size, 150 x 150 x 530 mm or 200 x 200 x 300 mm. The test specimens were moist-cured for 7 days, air cured (under vinyl sheets) for about 45 days and then installed at the exposure sites.

Evaluation Criteria

The evaluation criterion used was somewhat similar to that used by the U.S. Corps of Engineers, i.e. the determination of the relative dynamic modulus of elasticity and weight loss during the tests.

Type of Concretes Installed at the Exposure Sites

The water-to-cement ratio of air-entrained concrete ranged from 0.50 to 1.0 with entrained air contents ranging from 0 to 7 per cent. The slumps of the concrete ranged from 40 to 100 mm.

The results of the investigation to date have been disappointing as judged from the following conclusions drawn by Shimada et al (22).

1. In the exposure tests, the specimens with less curing days and lower curing temperatures showed larger relative dynamic modulus of elasticity. These trends were the inverse of the results from accelerated tests. As the possible explanation, it could be cited that strength development of concrete during the exposure period had substantial effect on relative dynamic modulus of elasticity.

2. Regarding the effect of the quality of aggregates, the results of exposure test showed that the resistance of concrete is lower as water absorption of aggregates is higher.

3. The effect of the type of cement and the water-to-cement ratio on the resistance of concrete to frost damage was not observed in the exposure tests. However, when sufficient water was supplied, trends similar to the results from accelerated tests were observed after one winter.

Field Exposure Site at Tianjin Harbour, North China

The Tianjin Harbour Exposure Site is located in North China. The daily mean temperature at the site is -4.1°C in the coldest months. According to Chen Boqi et al (23), the number of freezing and thawing cycles under normal conditions for the concrete exposed in the middle of tide range were 82 per year.

The investigations at this exposure site were primarily concerned with determining the effect of concrete materials and cover to steel on the corrosion of reinforcing steel. The reinforced concrete specimens used were rather small, 160 x 210 mm in size, with thickness varying from 36 mm to 106 mm. The water-to-cementitious ratio of concrete ranged from 0.65 to 0.45. Both non-air-entrained and air-entrained concretes were used with cementitious factors at 400 and 500 kg/m^3. Several concrete mixtures incorporated fly ash and some were made with portland/blast furnace slag cement.

The results of the investigations after 10 years of exposure only confirmed the previously known facts that thick covers, low water-to-cementitious materials ratio, and long curing periods before exposure were significant factors in reducing the corrosion of reinforcing bars. The only significantly new information was that the failures or damage occurred mostly above the average high tide level.

CORRELATIONS BETWEEN LABORATORY ACCELERATED FREEZING AND THAWING TEST RESULTS AND THOSE OBTAINED FROM TESTS AT NATURAL WEATHERING SITES

Several researchers have attempted to correlate results of laboratory freezing and thawing with those obtained from exposure of large test specimens at natural weathering sites such as Treat Island in the U.S.A. or Ishikari and Bibi in Japan. But these attempts have been largely unsuccessful. According to Mather (17), the paper by Kennedy and Mather presented the first- and last- serious attempt to correlate results obtained in the laboratory accelerated freezing and thawing tests and those obtained from the observation and testing of specimens exposed at Treat Island, Maine. In their paper Kennedy and Mather (24) stated:

> "The attempt to correlate accelerated laboratory freezing and thawing with natural exposure at Treat Island by comparing test results on 48 combinations of aggregate made into concrete with the same water-cement ratio, air content, and consistency, showed that each aggregate combination behaves in an individual manner in each exposure. The two exposures are different in manner and tend to accentuate different physical and chemical characteristics of the materials, thereby leading to dissimilar results. If prediction of behaviour in one exposure from behaviour in the other is to be made, all the differences in materials and exposures must be taken into account".

It appears that in some areas of research and development, progress is very slow, and the above statement concerning the correlations mentioned above and published in 1953 is equally valid today.

CONCLUDING REMARKS

1. Recent research and development in concrete materials in the U.S.A., Canada and Scandinavian countries has made it possible to make portland cement concrete which performs excellently in accelerated laboratory freezing and thawing tests, and when exposed to severe weathering conditions at natural weathering exposure sites such as Treat Island, Maine.

2. Of all the available accelerated freezing and thawing tests, ASTM Test Method C 666 appears to have gained the most acceptance worldwide. The Powers' dilation method and the RILEM critical degree of saturation method though elegant and promising have not been adopted by many laboratories in North America and Europe.

3. Investigations performed at CANMET in the area of superplasticizers and supplementary cementing materials clearly demonstrate that freezing and thawing resistant concrete can be made using the above chemical and mineral admixtures, provided concrete is properly air-entrained, and has adequate air-void parameters.

4. Natural weathering exposure sites such as those at Treat Island, Maine, and Bibi and Ishikari in Japan provide excellent facilities for monitoring the long-term performance of concrete under natural freezing and thawing conditions, and the use of such facilities is encouraged. However, attempts to correlate the results of laboratory accelerated freezing and thawing with those obtained from the exposure of large concrete specimens at the exposure sites have not been successful, and the perennial efforts to establish such correlations should be abandoned.

5. There is a need to standardize the size, shape and location of test specimens at the natural weathering sites. Standardization is also needed as regards to the location of thermocouples in the specimens, length of exposure and criteria for the evaluation of test specimens before, during and after exposure.

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APPENDIX 3

RECOMMENDATIONS FOR DURABLE CONCRETE FOR STRUCTURES LOCATED IN THE MARINE ENVIRONMENT

TABLE 1. RECOMMENDATIONS FOR DURABLE MARINE CONCRETE

Author or authority	Exposure conditions	Cement type	Maximum water/cement ratio	Minimum cement content, kg/m³	Minimum strength, N/mm²
Tyler[40]	Tidal and above	—	0·45	390	—
Gerwick[41]	—	ASTM type II (tricalcium aluminate <8%)	0·44	390	25
CP 110[42]	—	—	0·45	330	—
FIP[43]	Tidal zone	tricalcium aluminate <8%	0·45	400	40
	Submerged		0·45	360	30
Det Norske Veritas[44]	—	tricalcium aluminate <7%	0·45	400	45

TABLE 2. SPECIFICATION FOR DURABLE MARINE CONCRETE

	CP110 (1972)	DnV (1977)	NPD (1977)	ACI (1978)	BS6235 (1982)	AUS. (Draft) (1983)	BS811 (1985
IX DETAILS	Cement Content kg/m^3 and (w/c) ratio						
tmospheric Zone	360 (0.50)	300 (0.45)	400 (0.45)	- (0.40)	400 (0.40)	400 (0.45)	400 (0.4
plash Zone	360 (0.50)	400 (0.45)	400 (0.45)	- (0.40)	400 (0.40)	400 (0.45)	400 (0.4
ubmerged	360 (0.50)	300 (0.45)	400 (0.45)	- (0.40)	360 (0.40)	330 (0.50)	400 (0.4
OVER TO REINFORCEMENT	All Values in mm for rebar and (p/s)						
tmospheric Zone	60 (60)	40 (80)	50 (70)	50 (75)	75 (100)	75 (100)	60 (60)
plash Zone	60 (60)	50 (100)	50 (70)	65 (90)	75 (100)	75 (100)	60 (60)
ubmerged	60 (60)	50 (100)	50 (70)	50 (75)	60 (75)	60 (75)	60 (60)
HLORIDE CONTENT	Values in % Cl^- by weight of cement						
einforcement							
BS12 Cement or similar	0.35	0.19	-	0.10	0.35	0.20	0.4
BS4027 Cement or similar	0.06	0.19	-	0.06	0.06	0.20	0.2
restressing Steel							
All Cements	0.06	0.19	-	0.06	0.06	0.10	0.1
RACK CONTROL — Max. C/W (mm)	0.3 or 0.004 x cover	by agrement	-	- -	0.3 or 0.004 x cover	- 0.004 x cover	0.3 -
Max. steel stress (mPa)	-	160	-	120	-	-	-

TABLE 3. TYPICAL CONCRETE MIXES FOR THE MARINE ENVIRONMENT

x and properties	Type of concrete										
	Structural, cast in place	Structural, lightweight	Structural, precast	Prestressed	Cyclopean	Heavy-weight	Cellular	Tremie	Tremie Grout	Gunite*	Grout intruded
ment:											
Type	II or III	II or III	II or III	II or III	II	II or III	II	II	II	II or III	II
Amount, lb§	658	658	658	705	240	658	658	705	846	705	705
regate:											
Coarse:											
Type	Crushed rock or gravel	Expanded shale, clay or slate	Crushed rock or gravel	Crushed rock or gravel	Cobbles quarried rock†	Iron ore or barite	None	Gravel	Pea gravel	None	Gravel†
Max. size, in.	$1\frac{1}{2}$	$1\frac{3}{4}$	$1\frac{1}{2}$	$\frac{3}{4}$	12, $\frac{3}{4}$	$1\frac{1}{2}$ to 3	.	$\frac{3}{4}$	$\frac{3}{8}$		$1\frac{1}{2}$
Amount, lb§	1,900	1,100	1,900	1,900	2,000, 700	5,500	. .	1,800	1,800		To fill forms
Fine:											
Type	Sand	Expanded shale, clay or slate‡	Sand	Sand	Sand	Sand	Sand	Sand	Sand	Sand	Sand
Amount, lb§	1,200	750	1,200	1,200	400	1,200	1,200	1,300	1,500	3,000	3,000
ter, lb§	290	290	290	280	130	300	300	380	400	400	360
rmal comp. strength, psi	3,500	3,000	4,000	5,000	3,000	3,500	1,000	4,000	2,500	5,000	4,000
x. strength, psi	5,500	4,500	6,000	7,000	4,500	6,000	1,500	6,000	4,000	6,000	6,000
dulus of lasticity, psi	4×10^6	2×10^6	4.2×10^6	4.5×10^6	.	4×10^6	1×10^6	4×10^6		4.5×10^6	
t weight n air, lb/ft³	148	105	150	155		230	50			150	
t weight ubmerged, lb/ft³	84	41	86	91	76	166	−15	84		86	88
dulus of upture, psi	500	350	600	700							
mp, in.	2–4	2–4	$1\frac{1}{2}$–3	$1\frac{1}{2}$–2	6	2–4	3	6–8	Fluidizing		
mixtures	Air entraining (optional)	Air entraining	Air entraining (optional)	Water reducing	Plasticizing, air entraining	Air entraining	Foaming or gas producing	Plasticizing, air entraining	Plasticizing	Set acceler-ating	Retarding and fluidizing

TABLE 4. TECHNIQUES FOR OBTAINING SPECIFIC PROPERITIES OF CONCRETE FOR THE MARINE ENVIRONMENT

Property desired	*Techniques*
Chemical durability	Aggregates sound, nonreactive, and abrasion resistant. Cement moderate or low C_3A (Types II or V); high cement factor; low alkali content. Low water/cement ratio (may be facilitated by use of water-reducing admixture). Avoidance of sharp edges; proper curing; avoidance of horizontal construction joints in splash zone; drying after curing; air entrainment; bitumastic or epoxy coatings.
Protection against corrosion of reinforcement	Adequate concrete cover. High cement factor; moderate or high C_3A in cement (Types I, II, or III). Fresh water in mix; limit on chlorides from any source; well-compacted, workable mix. Waterproofing coatings; no embedded copper, aluminum, etc., which will produce galvanic action.
Freeze-thaw resistance	Sound aggregates; air entrainment; low water/cement ratio; proper curing.
Abrasion resistance	No sharp corners. Well-compacted, workable mix; low water cement/ratio; hard, abrasion-resistant aggregate. Good finish; adequate cure; steel forms.
Explosion waves	No reentrant angles or holes.
Cavitation resistance	Minimum of projections and obstructions to flow; dense, hard concrete; aggregates selected for high bond (crushed rock).
Protection against marine organisms	Hard, dense surface. Well-compacted, workable mix; bitumastic coatings; high cement factor; low water/cement ratio.
Impermeability	High cement factor; well proportioned mix; low water/cement ratio; well-compacted mix; proper curing.
Heavy unit weight	Heavyweight aggregates, such as iron ore, barite, iron punchings.
Light unit weight	Lightweight aggregates such as expanded shale, clay, or slate *or* cellular concrete produced by foam or gas.
High strength	High cement factor; low water/cement ratio; well-compacted mix; proper curing. High-quality aggregates of relatively small size, e.g., $\frac{3}{4}$-in. maximum size of coarse aggregate.
Prevention of cracks	Well-distributed reinforcement; prestressing; low water cement ratio. Good finish; proper cure.

APPENDIX 4

AN APPLICATION OF BAYESIAN ANALYSIS TO UNDERWATER INSPECTIONS

Introduction

Information produced from an underwater inspection is highly subjective. This subjectiveness is due to numerous causes including, but not limited to, the limitations of the inspector (diver), the difficulty associated with using equipment underwater and the uncertainties associated with the use/abuse of the facility. Prior to, during and after an underwater inspection, subjective judgement must be applied in order to produce data which is meaningful to the facility manager.

Due to the impracticality and prohibitive costs associated with performing a complete and through inspection each time a facility is to be inspected, statistical analysis is used to select portions of the facility to be inspected. By selecting only a portion of the facility to be inspected certain amounts of risks are involved with regards to the accuracy of the information produced by the inspection. By using Bayesian analysis, a formalized, statistical method of updating risk analysis, updated information can be incorporated into the analytical model to ensure that the inspected portion of the facility is a true representation of the entire facility.

This paper presents an elementary explanation of Bayes' Theorem and suggests an applied approach to its use in the field of underwater inspection.

Background

Typical waterfront structures can have several thousand structurally significant members which require inspection to ensure their continued performance. Since the major portion of these members are located underwater, the cost for through inspections is considerable. In these days of tightening fiscal constraints, it is not economically feasible to thoroughly inspect all of the underwater structural members of a waterfront facility. Therefore it becomes imperative that owners and facility managers develop methods to accurately inspect their facilities at the most reasonable cost while obtaining information that is representative of the actual condition of the facility.

Sampling methods and decisions made with regards to inspection plans should be based upon statistical sampling techniques and the members of the structure selected for inspection can be selected on the basis of probability theory. The advantages of using statistical sampling techniques lies in the support of the laws of probability and with the objectiveness applied in determining the variability of the samples selected. The

utilization of statistical sampling techniques allow the required sample size to be determined by evaluating the alternatives between inspection costs and the required confidence level of the information obtained.

Once accurate information is obtained with regards to the actual condition of the facility, the reliability of the structure to continue to operate can be determined and the expected cost of maintaining the structure for future operations can be evaluated.

Information obtained during the conduct of an underwater inspection is filled with bias. Bias is introduced by the ability and attitude of the inspector, the capabilities of the equipment used, the sample size selected, the actual members chosen for inspection and numerous other factors that influence the inspection plan. Some of the bias introduced into the inspection plan is not necessarily bad. Intimate knowledge of the severe loading conditions which the facility may have been exposed to, known contaminates released into the nearby water and other pertinent information can be invaluable to developing an inspection plan which will provide accurate information regarding the actual condition of the facility.

Classical statistical techniques do not allow for the infusion of historic information into the statistical model around which the inspection plan will be developed. This condition requires the inspection plan to be developed continuously from a position of ignorance. Historic information can be used to develop statistical models if Bayesian theory is utilized. Bayes' theorem is a manner by which previous information obtained can be used to updated the statistical model of a facility. The resultant statistical model is one by which past events known to have occurred can be utilized to influence the inspection plan.

Numerous articles have been published which both exalt and condemn Bayesian techniques and it is not the purpose of this paper to add any additional fuel to this fire which exists between statisticians and engineers. The purpose of this paper is to purpose an application which, by nature, is based upon subjective decision and therefore should lend itself to Bayesian techniques of updating by previously learned experiences. This chore will be undertaken by first listing some ot the pertinent definitions and nomenclature which pertain to statistics and probability and are referred to within this paper. Next an elementary presentation of Bayes' theorem will be presented along with the derivation of Bayes Theorem. Finally, a narrative section will be provided to purpose the use of Bayesian updating

techniques into the decision making associated with developing an underwater inspection plan for a waterfront facility.

Definitions

Coefficient of Variation (COV) - It relates the standard deviation and the mean by expressing the standard deviation as a percentage of the mean value (i.e. standard deviation/ mean value)

Conditional probability - The probability of an event given that some other event has already occurred.

Confidence Interval - An indication of the range of the estimate being made. It is usually expressed in terms of the standard errors.

Confidence Level - A probability which indicates the confidence that the interval estimate will include the population parameter.

Equally likely events - Each event has the same probability of occurring.

Estimate - A specific observed value of a statistic. A point estimate is a binary value (i.e., right or wrong, in or out, present or missing). An interval estimate gives additional

information about the range of values in the estimate and therefore an estimate of the error involved in the sample.

Event - A subset or part of the population whose occurrence is of special interest.

Independent events - Two or more events are independent if the occurrence of one event in no way affects, or is affected by, the occurrence of the other events.

Inspection - The process of measuring, testing, examining or otherwise comparing part or all of a facility to determine conformance to a predefined standard.

- Baseline Inspection - A preliminary survey of the facility to determine the as-built condition and any obvious stratifications for the facility. The baseline inspection may reveal conditions that require the scope of the inspection plan to be changed in order to achieve the desired confidence level.

- Level I Inspection - General visual inspection. Involves no cleaning of the structural members and can be done quite rapidly. The purpose is to confirm as-built conditions, detect obvious damage, severe corrosion or extensive biological attack. The information provided may be used for possible stratification of the facility during the subsequent inspection.

- Level II Inspection - A close-up visual inspection. It involves cleaning of the structural members in order to expose the surface to be inspected. The purpose is to detect any surface damage that may have been hidden underneath the biofouling.

- Level III Inspection - Nondestructive testing techniques (NDT) and equipment (NDE) are used to detect hidden or incipient damage. The equipment and the procedures generally require more experienced personnel than Level I or Level II inspections.

Inspection by Attributes - An inspection by which the population is characterized by a particular feature or attribute. The attribute may be acceptance/rejection criteria or the presence or absence of some feature. Inspection by attributes requires a larger number of elements to be sampled while requiring the least amount of sophistication in the measurement technique.

Inspection by Variables - An inspection wherein the data gathered during the inspection contain a continuous range of values from some lower to some upper bounds (i.e., calipers, ultrasonic devices and depth gauges). This type of inspection normally requires a small number of samples in order to project the statistic onto the population.

Mean - The average value of all samples taken. It is calculated by dividing the sum of all values in the sample by the number of samples.

Mutually exclusive events - The occurrence of any given event excludes the occurrence of all other events. Mutually exclusive events have no points of the population in common.

Parameter - A characteristic of a population (i.e., the average value of all the piles in a facility)

Population - The set or list of descriptions of all possible outcomes of the phenomenon or process being studied.

Probability - The likelihood that an event will occur.

- Empirical probability - The ratio of the number of ways the event can occur divided by the total number of possible outcomes. This definition applies to a finite population in which all events of the population are equally likely to occur. Sometimes called the relative frequency or statistical definition of probability.

- Classical probability - A measure of the degree to which available evidence confirms a given hypothesis. Sometimes called the objective definition of probability.

- Subjective probability - a personal opinion, or "degree of belief", of the likelihood that an event will occur.

Sample - A portion or element chosen from the population.

Sampling Plan - The set of procedures that specify the number and method of selection of the elements that make up a sample. It should also include the criteria used to determine if the sample statistics are representative of the population parameters.

Sampling with and without replacement - Sampling with replacement implies independent events. When a sample is selected it is observed and then placed back into the population where it becomes available for selection in subsequent sampling. The population remains the same after each observation. Sampling without replacement implies dependent events. When a sample is selected and observed it is not placed back into the population. Thus the population changes after each sample selection.

Standard Deviation - The square root of the variance. The units of this statistic are the same as the units of the samples taken and are therefore used to describe the dispersion of the values of the items sampled.

Standard Error of the Mean - It is the standard deviation divided by the square root of the number of observations in a sample. It is the most often used definition for the standard error associated with a sample set.

Statistic - A characteristic of a sample. The statistic is calculated from the observations of some percentage of the

population. This characteristic can then be used to make a statement about the population.

Stratification - The process of dividing the facility into homogeneous groups to improve the efficiency and accuracy of the inspection.

Variance - A description of the dispersion of the data from some measure of central tendency. This statistic is calculated from the sum of the squared differences between the mean and each of the data values.

Nomenclature

$P(A)$ = probability that event A will occur

$P(AB)$ = probability that both events A and B will occur

= the intersection of events A and B

= the joint probability of A and B

$P(A+B)$ = probability that event A and/or event B occurs

= union of events A and B

= probability that at least one of the events will occur

$P(A|B)$ = the conditional probability that event A will occur given that event B has already occurred

Multiplication Theorem: $P(AB) = P(A|B)P(B) = P(B|A)P(A)$

The probability that A and B will occur is the conditional probability of A, given that B has occurred, multiplied by the probability that B will occur (or vice versa).

If A and B are independent events the theorem reduces to;

$P(AB) = P(A)P(B) = P(B)P(A)$

addition Theorem: $P(A+B) = P(A) + P(B) - P(AB)$

The probability that A and/or B will occur is equal to the probability that A will occur plus the probability that B will occur minus the probability that both A and B will occur.

If A and B are mutually exclusive, the theorem reduces to; $P(A+B) = P(A) + P(B)$

Bayes' Theorem

In the later part of the eighteenth century Thomas Bayes constructed an essay concerning the utilization of prior discovered information to the probability of occurrence. Bayes never published his essay which has, over the years, added more fuel to the fire concerning the controversy associated with his ideas set forth within the essay. Shortly after his death, the essay was discovered and transmitted to the Philosophical Transactions of the Royal Society where it was published in 1763, some three years after his death.

The information contained within Bayes' essay began to be scrutinized by the 18th century philosophers for it demonstrated how inductive reasoning could proceed within the then more acceptable, deductive logic of mathematics and thus provide a greater credibility to the "cause and effect" argument so prevalent in that day. The mathematicians of the day quickly applied Bayes' result and Laplace gave credence to the ideas by incorporating it into some of his work. The true controversy apparently did not develop until the middle of the 19th century when Boole first questioned the development of the Bayes axiom. Since that time the debate has raged, not over the essay itself but rather with the interpretations and extensions of Bayes' work.

Bayes' theorem provides a mathematical basis for relating the degree to which an observation, or new information, confirms the various hypothesized causes or states of nature. The theorem can be derived from the basic axioms of probability theory and, as stated above, has been proved to be correct and is not the subject of the controversy. The controversy has confined itself to the utilization and application of Bayes' proposed "subjective probabilities".

Derivation of Bayes' Theorem

Let $E_1, E_2, \ldots, E_N$ be mutually exclusive events whose union is the population. Let B be an arbitrary (observed) event in the population so that $P(B) \neq 0$.

Then $P(B) = P(BE_1) + P(BE_2) + P(BE_3) + \ldots + P(BE_N)$ (1)

From the multiplication theorem of probability theory we can write;

$P(BE_1) = P(B|E_1)P(E_1)$

$P(BE_2) = P(B|E_2)P(E_2)$

$\ast$

$\ast$

$P(BE_N) = P(B|E_N)P(E_N)$ (2)

Substituting equation (2) into equation (1) gives;

$P(B) = P(B|E_1)P(E_1) + P(B|E_2)P(E_2) + \ldots + P(B|E_N)P(E_N)$ (3)

The joint probability of events B and E_1 can also be written as;

$P(BE_1) = P(E_1|B)P(B)$

Thus;

$P(BE_1) = P(E_1|B)P(B) = P(B|E_1)P(E_1)$ (4)

Solving for $P(E_1|B)$ we have;

$$P(E_1|B) = \frac{P(B|E_1)\,P(E_1)}{P(B)} \ldots \qquad (5)$$

Finally, substituting equation (3) into equation (5) we have;

$$P(E_1|B) = \frac{P(B|E_1)\,P(E_1)}{P(B|E_1)P(E_1) + P(B|E_2)P(E_2) + \cdots + P(B|E_N)P(E_N)} \qquad (6)$$

Similar results hold for $E_2, E_3, \ldots\ldots, E_N$. Thus equation (6) can be generalized to gives Bayes' Theorem.

$$P(E_i|B) = \frac{P(B|E_i)\,P(E_i)}{\sum_{i=1}^{N} P(B|E_i)\,P(E_i)} \qquad i = 1, 2, 3 \cdots N$$

Application of Bayes' Theorem to Underwater Inspections

The Bayesian approach can have special significance when applied to underwater inspections where information is usually limited and very subjective in nature. In determining the parameters of interest during an underwater inspection the facility manager or the tenant may have some historic information upon which some bias should be based. The operational tempo or specific instances of loading may be known and should be considered when the inspection plan is being developed. The departure from classical statistical estimation of the parameters of the inspection would be justified in order to lend special attention to areas of the facility that are known to be more likely to require inspection than the facility as a whole. As additional information becomes available, for instance the results of the baseline inspection, that information can be used to update the parameter estimations made during the planning process.

The acceptance of a Bayesian approach can also be significant in as much as the judgmental information can be used in the calculation of relevant probabilities associated with the inspection results. These probabilities can be updated by the infusion of the inspection information as it becomes available.

This continuous influx of information would allow the inspection to be a dynamic process which would be more likely to give more accurate results as related to the actual condition of the facility.

The conduct of underwater inspections of waterfront facilities requires the combination of historic information, judgmental assumptions and current information in order to produce inspection information which accurately portrays the current condition of the facility. The significance of prior information is eminent and can be formally utilized by incorporating Bayesian statistics into the inspection planning process. The key point is that Bayesian methods afford a formal probability procedure whereby the inspection process can be continuously modified and updated by allowing the current information to impact the inspection parameters. The end result will be superior knowledge of the actual condition of the facility as opposed to random sampling techniques and the utilization of classical statistics.

A word of caution should be injected at this point to ensure that the prior information applied to the Bayesian approach should be provided by qualified individuals who have first hand knowledge of the actually conditions to which the facility has been subjected to and this information should be tempered with

sound engineering judgement and assumptions. The engineering judgement and assumptions should be verified by increased inspection effort if the desired reliability so indicates.

Conclusion

The differences between Bayesian statistics and classical statistics are fundamental and obvious. The purpose of this paper has not been to continue the argument between the Bayesians and those who proscribe solely to classical statistics but rather to point out a specific instance where subjective judgement is required and the impact of updating information can greatly enhance the accuracy of the assessment of the facility. A Bayesian approach to underwater inspection planning allows the establishment of a degree-of-belief whereas the classical approach will only lead to the relative frequency of occurrence of deterioration of the facility.

When used for the planning and conduct of underwater inspections, the Bayesian approach offers the following obvious advantages over classical statistical methods:

1. It provides a formal and accepted method for incorporating sound judgement and observed data into the sampling selection process.

2. It systematically combines uncertainties associated with randomness and those arising from errors of estimation and prediction and thus allows the recognition and minimization of these uncertainties.

3. It allows the systemic updating of the inspection parameters by information that is becoming available currently thus allowing inspection process become a dynamic process.

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APPENDIX 5

REPAIR OF CONCRETE IN THE MARINE ENVIRONMENT: CATHODIC PROTECTION Vs. CHLORIDE EXTRACTION

9th Australasian Conference on
Coastal and Ocean Engineering
Adelaide, 4-8 December 1989

Repair of Concrete in the Marine Environment: Cathodic Protection VS Chloride Extraction

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1. INTRODUCTION

The detachment of surface concrete from structures due to reinforcement corrosion highlights the problems of aesthetics, durability, and safety to the structure users, owners and the public. It is estimated in the United States that over the next ten years more will be spent on restoring or reconstructing structures than in building new ones (1). Additional hidden costs, including vacation of the structure during repair, substantially adds to the cost of renovation.

Traditional patch repairs to marine structures are expensive, and some uncertainty remains on their long term efficiency. The use, as patching mediums, of repair materials possessing different physical and chemical properties to steel reinforcement and concrete substrates in changing environments, portend the possibility of short term repair failures. Patch repairs tend to be labour intensive, bearing a high maintenance cost.

This paper discusses two corrosion mitigation techniques which reduce the labour intensiveness of repairs; namely chloride extraction and cathodic protection.

The chloride extraction (CE) process has recently been developed for site use. It is an electrochemical process whereby chlorides migrate under an electric current and its attendent potential difference towards a positive pole at the concrete surface. Upon chloride removal, the surface of the structure can be treated by applying a chloride barrier coating to minimise future ingress.

Cathodic protection (CP) is based on the principle of inhibiting corrosion activity by passing a small electric current onto the steel reinforcement causing it to become a cathode, thus creating a zero potential difference between points on the steel surface and hence arresting the corrosion process.

2. BACKGROUND TO CHLORIDE INDUCED CORROSION

Chlorides can be introduced into marine structures by a variety of means. During construction, chloride salts may contaminate the structure due to the use of salt contaminated aggregates, sands and mixing water. Formwork and reinforcement may collect surface salt deposits if left in a marine atmospheric or spray zone and would introduce chlorides into the concrete if unmasked prior to construction. In addition, calcium chloride has been widely used as a set accelerator. During the service life of a marine structure, chlorides may penetrate the concrete depending on the prevailing environmental conditions. The wetting and drying conditions of the splash zone are particularly aggresive where a high surface concentrations of chlorides are built up. As the surface is wetted, water borne chlorides are drawn into the concrete under the action of capillarity. This zone is typically a 20 to 50mm surface layer. Chlorides from this zone are transported to the steel surface by means of an ionic diffusion process.

If there are chloride ions in the pore water adjacent to the reinforcing steel above a certain concentration, the passive iron oxide film breaks down. Due to various oxidation and hydrolysis reacions, acid is produced in the pit thus formed (2). A corrosion cell is set up with an adjacent area of passive steel reinforcement acting as a cathode where oxygen is reduced and the anodic dissolution of iron taking place at a small central anodic area. Chloride contents sufficient to initiate corrosion may vary from 0.1 - 0.4, or more, percent by weight of cement depending on the cement used, the moisture content of the concrete, pore water pH and interaction with other contaminants (e.g. Sulphates or Carbonation) (3).

3. BACKGROUND TO CHLORIDE EXTRACTION

The principle of the method is to transport chlorides through concrete to the surface by electro-migration, using an applied electric field. A direct current is applied between the reinforcement, which acts as a cathode, and a temporary anode which is mounted on the concrete surface. The temporary anode consists of a lightweight steel mesh which is embedded in a conductive fibre. The reinforcement and the surface electrode are connected to the terminals of a low voltage direct current source and chlorides are driven into the conductive fibre.

When tests have shown that the chloride content of the zone under treatment has fallen to a sufficiently low value, the current source is disconnected and the chlorides are removed together with the conductive fibre. The duration of the process can span 3 to 8 weeks prior to the application of a chloride barrier coat.

A recent review paper on the technique is available in the technical literature (4).

4. BACKGROUND TO CATHODIC PROTECTION

Cathodic protection is based on the principle of inhibiting corrosion activity by passing an electric current onto the steel reinforcement, making it the cathode, hence slowing or halting the corrosion process.

Cathodic protection of steel in seawater has been practised since 1824 and, during the last 50 years, it has been extensively and very successfully used for the protection of steel structures in water and soils.

In the late 1950's cathodic protection systems were developed for the protection of reinforced concrete bridge decks. These impressed current CP systems used high silicon cast iron anodes in an asphalt overlay made conductive by the addition of coke. Anode modifications (to overcome weight and thickness penalties) consisted of platinum-clad niobium wire and graphite fibres layed into grooves and set in a conductive polymer grout. A recent review of CP systems in reinforced concrete structures can be obtained (5).

Again a direct current is applied between the reinforcement, which acts as the cathode, and a permanent anode mounted into or on the concrete surface. Anode systems have also been developed for vertical and soffit surfaces of reinforced concrete. Typical anodes have consisted of conductive surface coatings, conductive polymer mesh with gunite overlay, and titanium expanded mesh with gunite overlay.

The reinforcement and the anode are connected to the negative and positive terminals of a low voltage D.C. source providing a positive current.

5. DESIGN CONSIDERATIONS

Certain design aspects must be considered prior to the installation of a chloride extraction (CE) or a cathodic protection (CP) system. Some of these are discussed below:

a) Where reinforcement steel is electrically discontinuous, stray current corrosion can occur under the influence of the impressed current. This discontinuity needs to be identified and addressed.

b) Cathodic polarisation yields hydroxyl ions thereby increasing the pH of the pore water at the cathode. Susceptibility of the concrete to alkali - aggregate reactivity should be assessed.

c) Hydrogen gas can also be produced as a cathodic product on the steel reinforcement. This could lead to a reduction of the steel/concrete bond strength or hydrogen embrittlement.

d) The primary anodic reaction would be expected to produce products of low pH and chlorine gas. This reaction could induce disbonding at the anode/concrete interface. This is of importance in CP systems only.

6. CHLORIDE EXTRACTION vs CATHODIC PROTECTION

* CE is not ongoing - i.e. 3-8 week treatment whereas CP requires ongoing anode maintenance and monitoring of system performance.

* CP needs to be properly detailed to ensure adequate performance and for aesthetics, whereas the CE anode is temporary and less attention to detail is required.

* CE is more simple to apply - i.e. installation costs less.

  The CE anode mesh and cellulose fibres are throw-away items. Surface preparation is minimal and the application of the fibre is by spray.

  The permanancy of the CP system, the intensive surface preparation and application make a more complex and time consuming installation.

* Analysis of performance criteria for CP systems has not been comprehensively determined for reinforced concrete structures - i.e. criteria not fully developed, whereas the CE performance criteria is straight forward (i.e. chloride contents and electropotential measurements taken before and after extraction). The CE criteria can be supported by physical and chemical testing.

7. TYPICAL COSTING

Costings for this paper have been based on:

1. Recent experience on wharf structures.
2. Conservative remedial action.
3. Worst case assessment.
4. Present day costings.

We have assumed as our typical structure a 15 year old marine wharf with reinforced concrete decking and steel piles. The chloride induced corrosion has occurred throughout the soffit area. Extensive repairs would need to be carried out over approximately $100m^2$ with a total affected area of $1600m^2$ to be restored.

Fig. 1 shows a flow diagram which suggests possible repair procedures for a marine structure. Because CP and CE necessitate less breakout (to reinforcement surface and/or good concrete) than a patch repair, lower costs are therefore incurred for concrete repair in CP and CE options.

Table I gives an indication of the initial costs to be expected for the options of patch repair, cathodic protection and chloride extraction on the same structure.

Costings for cathodic protection systems have previously been given for bridge decking or parking facilities which are typically less expensive due to the uniform surface and regular shapes in comparison to a wharf.

The CP system on which our typical costing is calculated is a conductive polymer anode with a gunite overlap. The determining factor would be the service environment (splash, abrasion, etc.) found below a wharf.

Chloride Extraction or electro-migration as a commercially feasible technique has only been available over the last 12-18 months and the costings, though conservative, are less accurate than those for C.P. Fig. 2 is a schematic diagram of the CE technique.

Table II gives an indication of the expected on-going costs over a 20 year service life of the structure after repair. Some assumptions have been made regarding materials/equipment deterioration and these have been highlighted and taken into consideration.

We have assumed a 20% increase over the original spalled area ($100m^2$) which would need attention every 5 years. There are two schools of thought to the maintenance of a soffit area patched and suitably coated.

1. As a result of chlorides already in the structure, concrete damage due to corrosion of the reinforcement will continue to escalate despite the presence of the barrier coating.

2. As a result of a good barrier coating, the corrosion and subsequent spalling will diminish with time so that less maintenance will be required.

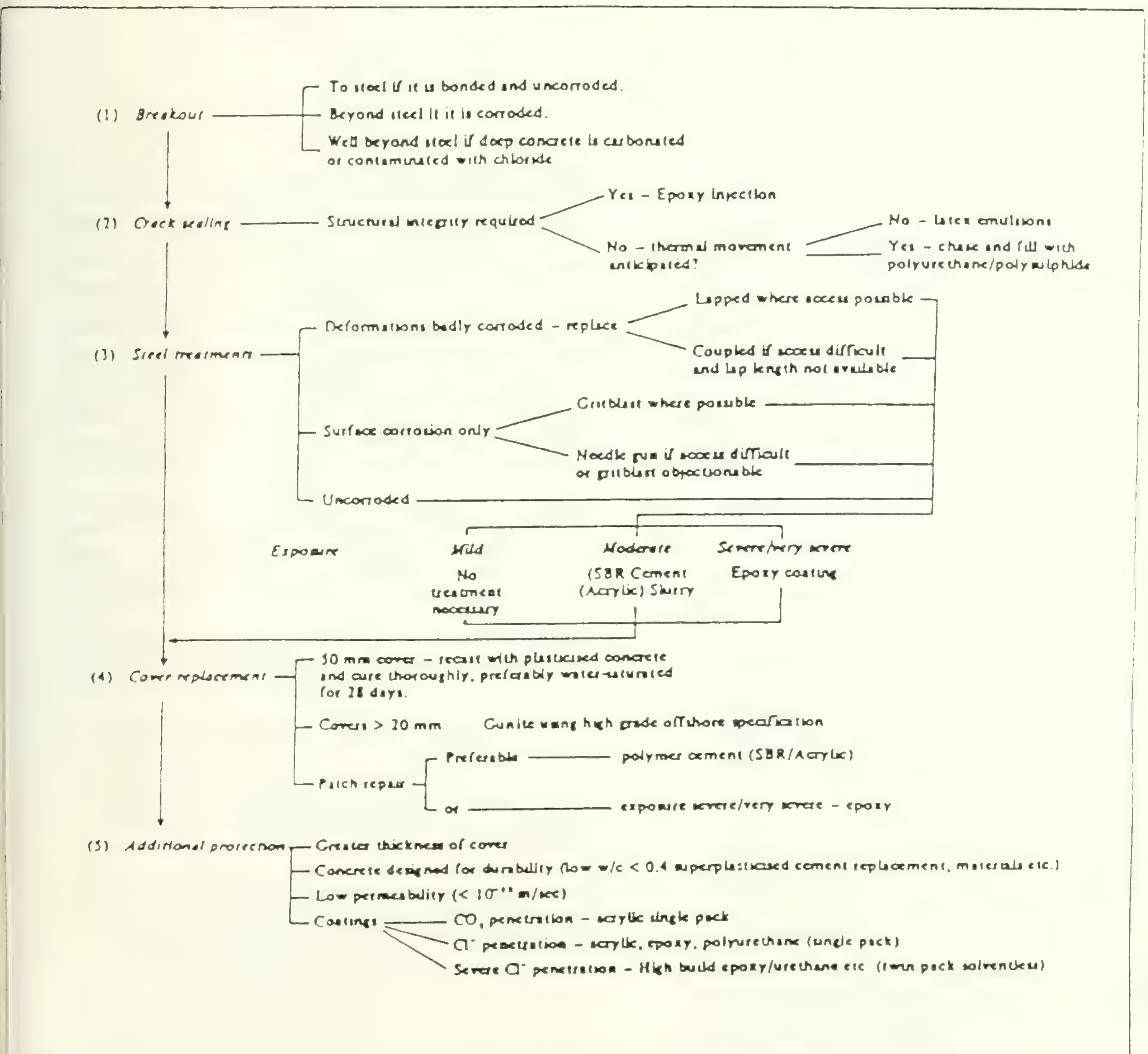
As our costings are based on a worse case assessment, we have opted for Item 1 above.

The initial cost benefit of the CE and patch repair over the CP application is demonstrated in Table I. There would be a 40-50% cost saving in installing the CE and conducting a patch repair in comparison with the CP system.

With regard to the on-going maintenance costs from Table II, CE and CP appear financially attractive over the patch repair option.

8. DISCUSSION

It has been reported recently that "early trial cathodic protection installations have failed dramatically", but that the need for remedial and restorative action is so great that use of cathodic protection systems on reinforced concrete structures are inevitable (5).



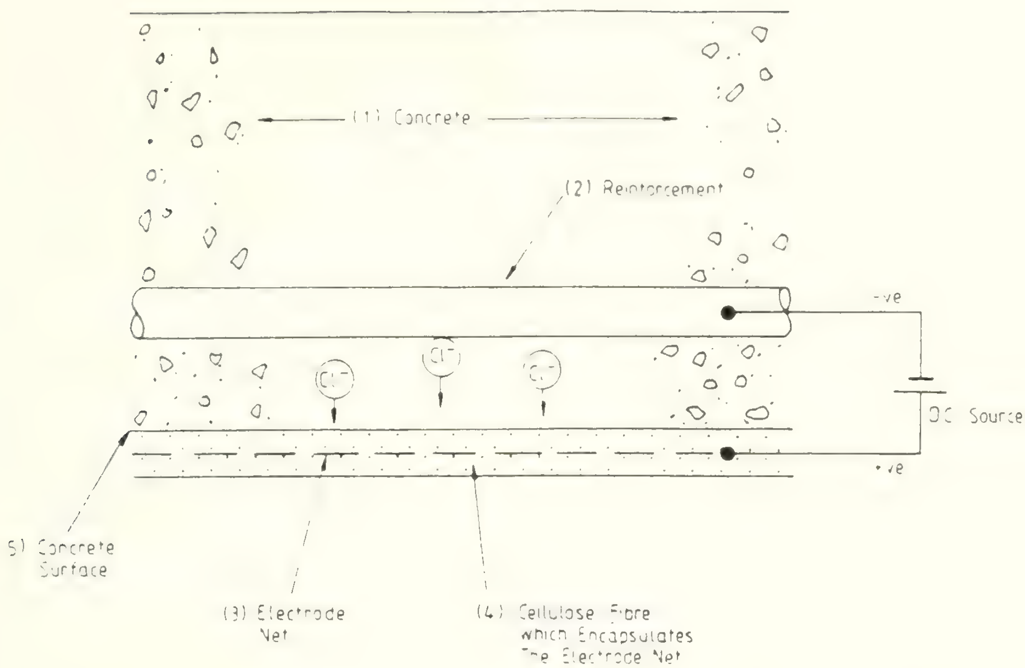


FIG 2 SCHEMATIC DIAGRAM OF CHLORIDE REMOVAL TECHNIQUE

A comparison of cathodic protection costings provided in this paper and that reported over the years suggests a significant discrepancy (6), (7). Previous costings for CP systems on bridge deck structures were some 50-60% less than that presented here. The reason for this discrepancy is the type and nature of structure being cathodically protected, the attendent problems of access, tidal windows for remedial work and the inconvenience in application, supervision etc. In this paper, we have based our costings on a severely corroded (5-10%), soffit area of complex shape, with a reinforced concrete wharf deck with cylindrical steel piles, a worst case assessment.

The chloride extraction technique is currently being trialled in Australasia and interim results suggest that it could be a viable remedial option for deteriorating structures under these environmental conditions.

It should be noted however, that any conclusions drawn regarding Chloride Extraction should be tempered with the knowledge that:

1. There has been a limited service history for structures and their response after treatment using CE.

2. Chloride Extraction can be capable of removing chlorides to <0.4% by weight of cement.

   Little work has been done to determine a chloride threshold to reinitiate chloride induced corrosion on structures previously subjected to CE.

9. CONCLUSION

This paper seeks to present succinctly some electrochemical techniques, Cathodic Protection and Chloride Extraction, which can be used in conjunction with normal concrete repair options to control corrosion in reinforced concrete marine structures.

Review of repair costs for a typical marine wharf with $1600m^2$ of soffit area subject to chloride induced corrosion have been addressed with repair option being patch repair, Cathodic Protection and Chloride Extraction.

10. ACKNOWLEDGEMENTS

The authors wish to thank Taylor Woodrow for permission to publish this paper and colleagues in the laboratory, particularly Miles Dacre and Warren Green.

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TABLE 1

TYPICAL COMPARATIVE COSTS FOR WHARF STRUCTURE

	ITEM	PATCH REPAIR	CATHODIC PROTECTION	CHLORIDE EXTRACTION
1.	Access (a) floating platform (b) access	50,000 6,000	50,000 6,000	50,000 6,000
2.	Remove defective concrete(100m^2)	137,000	46,000	46,000
3.	Surface preparation (100m^2)	6,000	6,000	6,000
4.	Reinstatement (shotcrete)	26,000	11,500	11,500
5.	Protective surface coating(1600m^2)	57,000	N/A	57,000
6.	Cathodic Protection Materials (anodes, half cells, electronics, etc)	N/A	211,000	N/A
7.	Cathodic Protection Installation (attach anode mesh, electrical wiring, installation monitoring equipment, apply shotcrete, commissioning) (16,00m^2)	N/A	144,000	N/A
8.	Chloride Extraction Materials (steel mesh, batons, cellulose fibre etc)	N/A	N/A	80,000
9.	Chloride Extraction Installation (attach steel mesh, apply cellulose fibre etc)	N/A	N/A	30,000
10.	Remove temporary Chloride Extraction installation (Monitoring, demobilisation etc)	N/A	N/A	20,000
	TOTAL INITIAL COST	282,000	474,500	306,500

TABLE 2

ON-GOING COSTS (20 YEAR LIFE)

Net Present Value (NPV) calculated assuming CPI 10% and interest rates (14%)

PATCH REPAIR

Initial Repair Cost	$282,000
Repeat 1, 2, 3, 4 every 5 years and assume increasing spalls by 20% / 5 years	
At 20 years (calculated to NPV)	$709,000
Repeat 5 at 10 years (calculated to NPV)	$ 34,200
TOTAL	$1,025,200

CATHODIC PROTECTION

Initial Repair Cost (Items 1, 2, 3, 4, 6, 7)	$474,500
On-going monitoring and maintenance (5% of (Item 6 & Item 7) at 5, 10, 15 years,	
At 20 years (calculated to NPV)	$ 31,900
TOTAL	$ 506,400

CHLORIDE EXTRACTION

Initial Repair Cost (Items 1, 2, 3, 4, 5, 8, 9, 10)	$ 306,500
Repeat 5 at 10 years (calculated to NPV)	$ 34,200
TOTAL	$ 340,700

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APPENDIX 6

INSPECTION PROCEDURES AND TERMINOLOGY

BUILDING 64

NAVAL AIR STATION

ALAMEDA, CALIFORNIA

INSPECTION PROCEDURES AND TERMINOLOGY

INTRODUCTION : This appendix defines basic inspection terms and describes the inspection procedure, equipment and techniques which are used to complete the January, 1990 underwater inspection of Building 64 located on the Alameda Naval Air Station.

DEFINITIONS : The following are definitions of standard levels of effort required to be exerted in each inspection. The Scope of Work for all inspections break down the total inspection effort into these levels and specifies the amount of work required in each level. The procedures prescribed for most inspections are commonly a combination of at least two of the levels of examination. The terms Level I and Level II, etc., are referred to frequently in the Scope of Work and in each inspection report. Their definitions are as follows:

LEVEL I : General Examination; This level of effort is essentially a "swim-by" overview, which does not involve cleaning of any structural elements, and can therefore be conducted much more rapidly than the other levels of examination. The Level I examination should be used to confirm as-built structural plans and detect obvious major damage or deterioration due to

overstress (ship impact, ice loading, ect.), severe corrosion or extensive biological growth and attack.

The underwater inspector shall rely primarily on visual and/or tactile observations (depending on water clarity) to make condition assessments. These observations are normally made over the total exterior surface of the underwater structure whether it is a quaywall, bulkhead, seawall, pile or mooring.

Visual documentation (utilizing underwater television and/or photography) may be included with the quantity and quality adequate for documentation of the findings which will be representative of the facility condition.

The results of the Level I inspections should be evaluated to determine the required amount of additional inspections which may be required. Level I inspections are recommended after the occurrence of any event which may be expected to cause damage or even undue stress on the facility. The use of remotely controlled vehicles is becoming more popular in the conduct of Level I inspections.

MODIFIED LEVEL I : This level of effort consists of a "swim-by" of every pile at an elevation of two to four feet

below mean low water line to detect any obvious gross or major damage.

LEVEL II : Detailed Examination; This level of effort is directed toward detecting and indentifying damaged/deteriorated areas which may be hidden by biofouling organisms or surface deterioration. At this level, a limited amount of measurements may be made. These data should be sufficient to permit estimates of facility load capacity to be made.

Level II examinations will often require cleaning of structural elements. Since cleaning is time consuming, it is generally restricted to areas that are critical or which may be representative of the entire structure itself. The amount and thoroughness of cleaning to be performed is governed by what is necessary to discern the general condition of the over facility.

Simple instruments such as calipers, measuring scales and ice picks are commonly used to take physical measurements. However, a small percentage of more accurate measurements may also be taken with more sophisticated instruments for several reasons. These will validate large numbers of simple measurements and in some hard-to-measure areas will actually be easier and faster to obtain.

Where the visual scrutiny, cleaning, and/or simple measurements reveal extensive deterioration, a small sampling of detailed measurements will enable gross estimates to be made of the structure's integrity. For example, on extensively deteriorated concrete piles with obviously corroded reinforcing steel, a small percentage should receive ultrasonic thickness measurements to determine the typical cross section profiles. The cross sections determined by these spot checks would be used to determine individual load capability which would then be extrapolated to obtain a "ballpark" estimate of overall facility load capability.

Visual documentation (utilizing underwater television and/or photography) should be included with the quantity and quality adequate to be representative of the range of facility damage/deterioration.

LEVEL III : Highly Detailed Examination; This level of effort will often require the use of Non-Destructive Testing (NDT) techniques, but may also require the use of partially destructive techniques such as a sample coring through concrete and wood structures, physical material sampling, or in-situ surface hardness testing. The purpose of this type of evaluation is to detect hidden or interior damage, loss on cross-

sectional area and material homogeneity. A level III examination will usually require prior cleaning. The use of NDT techniques are generally limited to key structural areas, areas that may be suspect or to structural members which may be representative of the underwater structure.

Visual documentation (utilizing underwater television and/or photographs) and a sampling of physical measurements should be included with quantity and quality adequate for documentation of the findings which will be representative of the facility condition.

APPENDIX 7

PAPERS COAUTHORED BY MR. FRED AICHELE

RESTORATION AND PRESERVATION OF MARINE STRUCTURES BY DIVERS

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INTRODUCTION

A large portion of the structure of America's port, harbor nd other marine facilities is below the surface of the water. matter what structural materials are used and what steps are ken to prevent deterioration of these structures there will be time dependent loss of integrity, much of it due to vironmental attack. As harbors have become cleaner in recent ars, increasing activity by marine organisms has caused damage structures. This is now particularly noticeable in older ructures which have a long history of use without the phisticated methods of preservation presently available.

Traditionally the solution to the problem of deteriorated rine structures has been simply to allow the structures to cay until they were unusable or unsafe and then replace them. is was a totally pragmatic solution in a world where there re no underwater inspection techniques capable of fully fining the problems, no underwater repair techniques capable producing repairs of high quality with assured results, and ere replacement costs were within the means of the responsible thority.

Replacement costs for marine structures have risen almost ponentially in the last decade forcing those responsible for e operation and repair of marine structures to look for other an traditional solutions. Fortunately, while costs of placing marine structures were growing so was the technology maintaining, restoring and preserving them.

As the majority of inshore underwater work occurs in depths one hundred feet or less, divers remain the primary means for ing this work. Unmanned work systems which eliminate exposure man to the hazards of the underwater environment and minate the decompression debt incurred by divers at depth se much of their appeal at shallow depth where depth related zard is minimized and the decompression debt build up is not ere. The dexterity of divers, which to date has not been eeded by mechanical or robotic systems give divers the edge effectively using tools to accomplish shallow underwater k.

This paper describes the means by which divers undertake the air and preservation of a common underwater marine structural ponent – pilings. Particular tasks are described in detail

s best showing how the use of divers for this type of work is ost effective and efficient. There is no implication that reservation and repair work on marine structures should be imited to any specific task or group of tasks.

ORK DEFINITION

As with any task the first step is to define what must be one. In the past indefinite and incomplete work definition has esulted in cost growths which have been, at least, an mbarassment to the authority responsible for the facility. eans now exist to define the underwater repairs required in uch detail that complete work specifications can be prepared nd detailed evaluations can be made by experienced technical ersonnel. In addition, post repair inspections can be made to ocument the exact conditions which obtain on the completion of ne repair. The primary tool used in work definition and nspection is television with non-destructive test techniques unning a close second. These tools allow divers working under ne direction of technical personnel responsible for the cility to collect data which can be used in the preparation of e work specification and can be made part of a bid package. A tailed discussion of television and non-destructive test chniques for definition of underwater work, progressing work d inspecting and documenting "as found" and "as repaired" nditions is beyond the scope of this paper which is concerned ly with the mechanics of specific underwater repairs. Suffice to say that such techniques exist and are extremely fective. As a matter of good business in contracting derwater repairs, the diving contractor who does the data thering for work definition and the inspection and cumentation work should be proscribed from bidding on the pair work.

In preparing work specifications and bid packages it is rongly suggested that the contents be limited to as detailed a scription as practical of the work to be done. Inclusion of a rge amount of detail in the package allows the diving ntractor to prepare an accurate bid and eliminates the mptation to either take short cuts later or to include a nerous contingecy allowance to account for unknowns. In so r as possible the work package should be limited to a scription of the work to be done. The temptation to tell the ntractor precisely how to do the work should be avoided. The lerwater contractor is a specialist at his work and will have formed similar jobs for a number of customers. He has seen a riety of conditions and has learned by experience what works l. A better, more cost effective result will generally be ained if the details of how the job is to be done is left to discretion of the diving contractor. Neither facility agers nor the divers hired to do the survey work should do

e engineering. All surveys should be reviewed by a qualified gineer or engineering consultant. During the survey, areas quiring repair should be marked and identified. Prints should thoroughly and exactly marked indicating repairs to be made. ney spent on surveys and engineering is returned many times in e long term because underwater contractors are given the tools thoroughly plan and accurately bid the work. The above does t imply that the contractor should not be required to prove to e contracting authority that his methods are sound.

The work described hereinafter, piling repair, is common in intaining marine structures. The variety of techniques and chnologies are representative of what divers can do in the storation and repair of marine structures.

PES OF DAMAGE TO PILINGS

Typical damage to underwater structures can be categorized :

1. Design Discrepancies
2. Construction Discrepancies
3. Deterioration
4. Collision/Impact Damage

Timber, steel and concrete pilings are all subject to duced life when used in the marine environment because of the rsh conditions that exist.

Problems which affect each type of pile are:

Wood Pilings:

1. Marine borers,
2. Overdriving,
3. Above-water rot,
4. Improper preservation,
5. Impact.

Concrete Pilings:

1. Insufficient concrete cover over reinforcement,
2. Improper or lean concrete mix,
3. Poor workmanship,
4. Chemical reaction from seawater or pollutants,
5. Impact.

Steel Pilings:

1. Corrosion,
2. Impact.

?ESERVATION OF WOOD PILINGS

Damage to wooden marine structures in the U.S. is estimated : $500 million annually. The preservation of wood pilings has ecome more common in recent years as underwater deterioration is ceased to be affected by the "out of sight - out of mind" ndrome.

While the specific techniques and tools used for preserving lings are widely varied, barrier wrapping has become the cepted means of preserving wood piles from ultimate loss from terborne worm (marine borer) attack. The tidal zone is rticularly susceptible to marine borer attack because in this ea creosote wears away first. The resulting vulnerability can reduced by surrounding the pile with a tightly fastened pervious barrier wrap.

The "barrier wrap" uses the "suffocation principle". A thin yer of water entrapped between the pile and the wrap becomes pleted of oxygen, suffocating the oxygen-dependent organisms. e wrap itself resists penetration and further attack. Two terials are being used: polyvinylchloride (PCV) and lyethelyne (PE).

C MATERIAL

PVC wraps, either manufactured or custom made, usually are mil thick with a minimum tensile strength of 2000 psi and are ecially formulated for longtime durability. Conservativly, fe expectancy of PVC wraps should be fifteen to twenty years, cepting damage from floating debris or ship impact. Where eosote oozing is probable, a polyethylene liner should be used prevent creosote attack on the interior of the wrap.

Cleaning and preparation of the pile surface must remove all ojections that could penetrate the installed wrap (nails, ts, encrustations, etc.). Cleaning may be accomplished by nd scraper, pneumatic or hydraulic tools, or by high pressure isting. Large cracks or deteriorated areas should be cleaned rotted material and filled with an epoxy or hydraulic cement ler.

There are two types of PVC wraps in use. The first is stalled in sections 8 to 10 feet in length. Each section has wood strip approximately one inch square or round attached to h side of the PVC wrap. The wood strip normally is a pical hardwood of the apitong family (apitong, gurjun, uing and yang). Besides resisting marine borers, these woods hstand the torques imposed while ratcheting to tighten the p. The barrier material is placed around the pile and the den strips brought together. A specially built racheting l is inserted between the wood strips and the barrier erial is wound tightly. The wood strips and barrier wrap

terial are fastened to the piling using 4" aluminum nails. wever, aluminum nails should never be used in pilings that ve been treated with copper-arsenate preservatives because the uminum will deteriorate rapidly. Where these preservatives e used steel nails will suffice. The ends of the wraps in the ter-tidal zone can be banded with aluminum or plastic banding terial or filled with a polyurethane foam seal. The ends of e wraps should overlap a minimum of one foot and be nailed d/or banded. Bottom wraps should extend below the mud line at ast 12 inches.

The second type of the PVC wrapping is a tube that zips gether after being cut to the desired length. The excess terial is folded over and held in place with plastic ties aced every three feet. The top and bottom of the wrap are aled in the same manner as described above.

LYETHYLENE WRAPS

Polyethylene (PE) wraps are normally 150 mil thick and are ailable in lengths up to 14 feet. Longer sections are nerally too awkward for divers to handle.

The same pile preparations apply to this type of wrap as to e 30 mil PVC wrap. Polyethelene wraps can be purchased either extruded pipe that has been cut and drilled or as flat sheets lled for nailing. The major difference between the two types cost. Using sheet can result in a savings on materials of at ast 20% to 25%. No matter which is chosen, for best results ps should be custom fitted to the pile. This is done by ering each wrap so that when it is installed it will be free the puckers between the nails known as "fish mouths" .

The PE (150 mil) wraps are primarily used in the inter-tidal e because of their toughness. Foam seals may also be used on top, bottom and vertical seam, although if installed perly with a 6 inch overlap, the seal is redundant.

The material is placed around the pile and a nail is tially driven in to hold the wrap in place. The diver then ces his first wrap wrench in position and snugs up the wrap, cking the vertical edge for overlap and proper position. itional wrap wrenches are positioned approximately one foot rt and all tightened. The number of wrenches used and their ign are the responsibility and option of the underwater tractor. After tightening, the wrap is nailed. As with the wrap, aluminum nails are used unless copper arsenate pounds have been used in wood preservation. The wrenches are ositioned and the process repeated until the wrap is pleted. The bottom and top are done last. A three inch ter to center nail pattern should be used and washers of the

me material as the nail placed under the nail heads to help read the pressure on the wrapping material and prevent the ap from pulling over the nail head if the nail is driven too rd.

A combination of PE and PVC wraps are being used in several rt facilities on the West Coast. The PE wraps are used in the ter-tidal zone and PVC on the remainder of the pile. The type activity at the facility determines where the PE stops and e PVC starts. For instance, if the facility sees a lot of tivity from deep draft vessels and tugs, then the 150 mil wrap ould be extended further below the surface as to prevent bris pushed from propellor wash from tearing the wrap.

OD PILING RESTORATION

Repairs by divers are made primarily to repair damage from rine borers or impact. If inspections reveal that structural tegrity is threatened, decisions must be made as to the type repair system to use. Repairs are generally made with ncrete using forms made from plywood, oil drums, steel sings, nylon fabric with zippers, and reinforced fiberglass.

There have been good results in the past several years in storing all types of pilings using reinforced fiberglass forms jacketing.

The area to be jacketed with concrete must have the same reful preparations as for wrapping. If the piling is teriorated to the point where it does not support the ructure, the damaged section is cut out and a jack installed load the piling its design weight. Using welded wire terial (W.W.M.), a cage is placed around the area to be keted to within 6" of the ends of the jacket. Stand-offs are sitioned to keep the W.W.M. away from the piling and to ensure t after the concrete is poured there will be at least 3" of ncrete between the W.W.M. and the form. If the jacketing ends to the bottom, a base plate to hold the bottom of the m is not usually needed. If the repair stops mid-way, a form e plate is installed with squeeze clamps to hold it in ition. The fiberglass form is then installed and buttoned After it is centered, an upper squeeze clamp is installed maintain alignment. If the form is very long, form supports be needed. After their installation, pumping can begin.

Pumping can be accomplished with either a tremie pipe or a ve at the base of the form. We have found the tremie pipe hod to be the easier. For a fairly long pour, or with very tle space between the top of the form and the structure ve, a tremie pipe can be installed at the same time as the

W.M. and left in place. Two inch, Schedule 80, PVC pile ould be used for tremie. No matter what method is used, the remie pipe must be positioned allow the concrete to drop not ore than 4" to 6" on the initial pump or concrete separation ll occur at the base of the jacket.

As the form fills, the tremie pipe is extracted slowly, eeping it at least one foot below the level of the concrete ing pumped. To ensure a good pumping flow and the filling of l voids, we usually use an 8 sack mix with 3/8" aggregate and lditives and with a 5" to 6" slump. This mix gives a 4000 psi rength in the 28 day test. After pumping is complete, the rms should remain in place for at least 24 hours, although it y be possible to strip them in as little as 12 hours. Once e forms and hardware are removed, the top of the jackets may finished off with an epoxy or hydraulic quick set cement. pering the tops is preferred.

To obtain the desired results of the repair, it is critical at the work be inspected at various stages of completion and reful attention be given to the repair and concrete design.

ESERVATION AND REPAIR OF CONCRETE PILINGS

At the present time, there are no specific systems in use r the sole purpose of preservation of concrete pilings.

The restoration methods for concrete pilings are similar to ose used for wood pilings. The major difference is in the le preparation. Concrete pilings must be thoroughly cleaned, l loose concrete removed, and scale and rust removed from posed reinforcing. Reinforcing must be repaired if damage is cessive. Placement of good quality concrete and proper stallation will assure a proper repair. Whenever possible, ncrete repairs should be shielded from seawater with permanent rt forms.

SERVATION AND REPAIR OF STEEL PILINGS

The systems used for preservation of wood pilings can also used on round steel piles. Since corrosion cannot continue the absence of oxygen, the suffocation principle applies. major problem in preservation is that steel pilings come in ious shapes which include such things as the railroad tracks er Fisherman's Wharf in Monterey. Custom fitting of barrier ps is thus particularly important for steel pilings.

The same repair principles and methods apply to steel piling to concrete and wood.

NCLUSION

The preservation and repair of that common component of rine structures, pilings, has become a routine task for divers recent years. It will be many years before the savings alized by using these techniques are quantified. The lesson clear now. Diver repair of major marine structures is both chnically and cost efficient. Designers, planners and those o maintain marine structures should be aware that divers are fective and should plan for their use in maintenance programs.

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MODERN INSPECTION TECHNIQUES IN PORT MAINTENANCE

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MODERN INSPECTION TECHNIQUES IN PORT MAINTENANCE

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I. WHAT'S THIS ALL ABOUT?

This paper addresses inspection of underwater structure of port facilities to acquire information which, when subjected to engineering analysis and confirmation, allows planning of short and long term maintenance. Inspection is the systematic collection of data for engineering analysis by visual, photographic, non-destructive test or other means employing tools and techniques suited to the structure and the environment.

II. WHY INSPECT?

It is well established that in the operation of any equipment or facility a planned and meticulously carried out maintenance program is cost effective because it results in significant long term savings and increased operational availability. Rising costs associated with maintenance of complex facilities demand that maintenance dollars be spent carefully to realize the potential long term savings and increased operational availability. Spending maintenance dollars carefully means precise definition of the work to be done and its scheduling or what work can be deferred. The maintenance engineer or planner cannot afford to plan his budget without the best information that is on hand or obtainable.

The first step in a cost-effective maintenance program is an inspection program which quantifies the conditions of the plant and permits engineering analysis and confirmation of those conditions. A program may be a onetime effort aimed solving a particular problem or it may be a long term program which develops history to predict or avoid problems to enhance maintenance planning and budgeting.

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Any inspection can do at least the following things:

* Verify the integrity of the structure,
* Determine the necessity for repair or more detailed inspection,
* Determine the scope of repairs,
* Allow the preparation of detailed repair specifications.

A more comprehensive long term program can add to that list:

* Development of planned maintenance schedules,
* Reduction of shutdowns,
* Prediction of incipient failures,
* Feedback into future design.

III. INSPECT UNDERWATER? ARE YOU KIDDING?

The above water portion of a structure presents well understood inspection, work definition and maintenance problems. The portion below water is a very different case. The incentive for the development of underwater inspection technology came from the offshore oil industry where structures of radical design are exposed to extreme environmental conditions. The inspection technology developed offshore has widespread application to port facilities and other civil structures. With maintenance and repair costs for aging structures rising and technology available to gather data for engineering evaluation and confirmation, there is no longer a reason for the "out-of-sight, out-of-mind" attitude that has prevailed in the inspection of underwater civil works.

Use of offshore underwater inspection technology in the apparently more benign environment of harbors and inland waters, has great appeal. The appeal stems from the realization that underwater structures can be inspected effectively and that a good inspection program can result in better maintenance dollar use. There is not a direct correlation between the offshore and inland environments. Differences range from materials - wood is not used offshore - to environmental conditions like the turbidity of the water. There are ongoing efforts to adopt offshore techniques to the conditions found in ports and harbors and to develop technology that works where existing techniques are not adequate, .

This paper discusses three primary methods of underwater inspection:

* Visual,
* Photographic, including stereo-photography and video and,
* Non-destructive testing.

Also discussed are the methods of deploying the inspection tools and means for collecting data.

IV. WHAT ARE WE INSPECTING FOR?

The goals of an inspection program should be defined in as much detail as practicable prior to initiating the program. In defining inspection goals it should be kept in mind that inspection is purely a collection process to provide raw data for engineering evaluation and confirmation.

Goals may be quite specific for small jobs, such as:

> "Conduct a video inspection of 15 batter piles between the 300 and 350 foot marks on pier N to allow determination of the damage done during the berthing of M/V NEVERSAIL."

Or they may be quite general for larger jobs:

> "Inspect using appropriate photographic and non-destructive test techniques all underwater structure on Piers A, B, and C in order to provide data which will define the necessity for repairs necessary to maintain 80% of design strength for five years."

More comprehensive the programs have greater potential for long range savings. Savings accrue from developing a facility history to predict maintenance requirements and from developing substantive information for feedback into future designs. In the initial definition of inspection goals an underwater contractor or consultant who specializes in underwater inspection can be invaluable in helping the Port Authority to define goals that are within the state of the art and the ability of available contractors to achieve.

The choice of the appropriate inspection technology is extremely important. Techniques appropriate for one type of structure or material may be totally unsuited for another. Applying the wrong technology can be wasteful and frustrating, and can lead to the erroneous conclusion that underwater inspection is not workable. The same is true of the means of deploying and handling the inspection implements. For instance, remote operated vehicles (ROV's) which are very good in linear

or area inspections lose usefulness around and inside pier structures because tethers tend to become entangled in structure or debris. It is cost effective for Port Authorities learning the underwater inspection business to have expert assistance in selecting the inspection technology and the means of deployment. To ensure objectivity the underwater contractor or consultant who assists the Port Authority should normally be proscribed from bidding the inspection work.

It is strongly emphasized that the inspection process is simply data gathering. The utility of the data depends upon the subsequent engineering analysis and employment of the data. To ensure the proper data is obtained in a form which can be utilized engineers and managers should participate in inspection planning and technology selection. Engineers should participate because they will be doing the analysis; managers because they will use the data for a facility history or repair specification.

V. HOW CAN WE DO THIS INSPECTION?

In the recent past underwater inspection was limited to visual or touch inspection by divers. Often the divers were not trained to recognize or describe conditions found in terms meaningful to the engineer. The engineer was dependent upon inadequate or incomplete reports and was unable to evaluate completely the structure and write complete repair specifications. The ability to conduct more meaningful inspections was increased by the employment of underwater television and has been further enhanced by non-destructive test procedures specifically adapted for underwater structures.

A. Cleaning

All inspection techniques require some cleaning of the surface to obtain accurate observations and measurements. The degree of cleaning required is dependent upon the inspection technique to be used, the structural material and the type of surface coverage. With harbors becoming cleaner there is an increase in the quantity of marine life which must be removed from port facility structures before cleaning. If the structure is heavily encrusted with marine growth or other surface cover cleaning in preparation for inspection may be more time consuming than the inspection itself.

Among the techniques used for cleaning surfaces for inspection are powered and hand brushes, scrapers and grinders. Needle guns and high pressure water or slurry

jets may also be used. Needle guns are not recommended as they tend to peen the surface of heat treated steel surfaces and to remove sound material on wooden and concrete surfaces. High pressure water jet pressures can be varied to suit the surface being cleaned and the thoroughness of the cleaning can be tailored to the inspection process being used.

B. Visual Inspection

While most inspections are conducted using aids for data gathering and the production of permanent records diver visual inspection remains an important procedure. Visual inspections are particularly good as initial inspections to locate areas of interest for later detailed inspection or testing. A major shortcoming of visual inspections is that reports are based on observations that are necessarily lacking in both precision and completeness. No two observers of the same defect, no matter how well trained, will make the same report. It is difficult for divers to make accurate measurements. In water with low visibility the diver may be required to use touch as a primary sensor with a decrease in the accuracy of his observations.

To ensure that a visual inspection is as complete as possible and not conducted in a random or haphazard manner the diver should be provided with as much information as possible about the structure he is to inspect and conditions he can expect to find. A surface supplied diver with a voice communications system rather than a SCUBA diver should make the inspection. The diver should report his observations in a standard prearranged format and vocabulary. The tender should record exactly what the diver says. A better inspection will be obtained if the engineer who will use the inspection results is on hand to hear the reports, to ask questions while the diver is in the water and to debrief the diver upon his return to the surface.

A visual inspection alone can provide useful data, if carefully prepared and carried out, but it provides no definitive record and is a crude and heavy handed way of producing results that are at best marginal.

C. Photographic Recording

Photographic recording means the use of chemical or electronic imaging techniques to produce a permanent visual record on film or tape. Permanent records are objective quantifiable data to document the condition of the structure, to facilitate evaluation of its current condition by qualified people and to provide a record

for determination of the rate of deterioration. Still photography, videophotography or a combination may be used.

1. Still Photography

Still photography may be used in either general survey or in detailed inspection of a particular item. Because of the additional information produced color photography is usually preferable to black and white. Both slides and prints are useful; which is chosen depends upon the use to be made of the record. The same general principles of photography regarding film speed, focus, depth of field and exposure control apply underwater as on the surface. In general, a slower (less sensitive) film should be used for inspection work because a sharper image is produced. A wide angle lens is useful as it results in a greater depth of field and a better view at the close ranges usual in underwater inspection. As both the slow film and wide angle lens require a high light intensity supplementary light should be used. Synchronized electronic flash is the preferred method of providing supplementary light. To make optimum use of available light the distance between the object being inspected and the camera should be minimized and the lighting placed to reduce flare, excessive contrast and backscatter. For most underwater inspection work the 35mm format is preferred. The utility of still photography in water with large amounts of suspended particulate matter may be limited. The ultimate usefulness of the inspection photograph in analyzing the condition of the structure is dependent upon the skill of the photographer. It is cost effective to plan underwater inspection photography carefully and to employ diver-photographers who are experienced in making photographic inspections.

2. Stereo-photography

Stereo-photography, wherein overlapping images are taken with identical lenses with a known separation, produces a three dimensional effect and allows accurate three dimensional measurements to be made by photogrammetry. The accuracy of the technique is highly dependent on the equipment, its condition and the skill of the photographer. The primary advantages of stereo-photography and photogrammetry are that they reduce lengthy, relatively inaccurate diver measurements and increase the amount of information obtained over other techniques for making precision measurements of conditions visible from the surface of the structural component. In harbor inspections stereo-photograpy and photogrammetry are valuable primarily in making small area inspections to examine conditions which have been

located by a general survey and require closer examination. These techniques have the disadvantages of requiring special equipment for viewing and complex computer based techniques for complete analysis.

The turbid waters found in most harbors present a problem for both still and stereo-photography that can be eliminated by the use of a clearwater box between the camera and the object being photographed. This technique is especially useful in stereo-photography where clarity is important for photogrammetry.

3. Videophotography

The most common tool used for underwater inspection is the video camera, monitor and recorder. The video camera presents real time information to operators and technical people on the surface. It essentially puts the eyes of the technical people underwater and allows them to direct the acquisition of particular data and to vary the conduct of the inspection in response to emerging information. The videotape provides a valuable permanent record when properly marked and narrated.

Both color and black and white television are used for underwater inspection. The color differentiation and detail make color systems particularly suitable for general survey and inspection. The black and white cameras offer greater resolution and are good in areas where great detail is required or there is little light. Low light level cameras such as the silicone intensified targeting type give good results in the dark waters encountered in most harbors. Such cameras can produce usable high resolution results in light levels as low as .0001 foot-candles; whereas vidicon tubes, the most common type, produce usable results in a minimum light level of one foot-candle. Because all types of cameras have been miniaturized and have automatic focusing features they can be deployed in "hands off" modes, such as being mounted on diver's helmets or on remotely operated vehicles.

Stereo-video systems are available. They are generally used for applications where the three dimensional effect is particularly important as in manipulator and vehicle control, and not for inspection.

As with still photography supplementary lighting is often desirable with underwater video, particularly when vidicon tube cameras are used. Mercury vapor and thalium iodide high intensity light sources are suitable for use with black and white cameras. They are not suitable for use with color systems because of their intensity in the blue-green portion of the spectrum.

For color television underwater tungsten and quartz halogen lights are preferred because their high red light content compensates for early red light absorption in water.

One of the major benefits of video inspection is found in the monitoring and data recording. The real time monitoring of the inspection allows emphasis to be directed towards areas of interest as they are discovered by the general survey. Recording of the inspection allows the material to be played for detailed study or for the information of senior managers. The VHS recording format has proven satisfactory for all types of inspection service. The video system should always include equipment for marking the tape with supplementary information and a timer which prints time and date information on the screen.

The value of a videotape is increased greatly by an accurate narrative. The purpose of the narrative is to orient the viewer and describe in general terms what he is seeing. As in visual inspection a standard vocabulary should be used. While reference should be made to specific conditions noted, the inspectors should attempt only to describe the condition and not to preempt the engineers' work of diagnosing the condition.

In all types of inspection where the technique is primarily visual a size reference is extremely useful. Such a reference is particularly useful when a permanent record is to be kept for determination of condition changes with time.

D. Non-destructive Testing

Inspection of the surface to a finer degree than can be obtained with either the human eye or surface imaging or inspection of the interior of the structure requires the use of non-destructive test (NDT) techniques. Because of the rapid progress being made in the field of applying NDT underwater, a detailed discussion of system capabilities is likely to be immediately outdated. Accordingly, no such description will be attempted. Rather, a general description will be undertaken of those techniques are being used offshore and in harbors. A general principle in underwater work that applies particularly to underwater NDT is that as much equipment and analysis as possible should be done on the surface. The diver has enough to do to operate the instrumentation which collects the data. Far better inspections result when the diver is simply an instrument for feeding data to a NDT technician on the surface.

1. Radiography

Radiography is one of the most common NDT techniques used on the surface for detection of interior volumetric flaws. Radiographic techniques have had limited use underwater. The primary use has been in dry hyperbaric welding of pipelines. There has been some radiographic inspection of flat plate structures using gamma emitting isotopes, but the technique is not generally employed. Radiography has the disadvantages of having radioactive materials at the inspection site and of requiring highly trained and well-qualified surface and diver technicians.

2. Magnetic Particle Inspection

Magnetic particle inspection can be used to detect discontinuities in the surfaces of ferromagnetic construction both on the surface and in the water. In underwater inspections the ferromagnetic particles are coated and suspended in a dye, usually water based, which becomes fluorescent under ultraviolet light, usually with wavelengths in the 3000 to 4000 angstrom range. The dye containing the particles and the light are generally deployed by means of an easily handled gun. A magnetic field is induced by application of a permanent magnet, electromagnet or current prod making the test piece the conductor. magnets with pulls of 15 to 150 pounds and low voltage (10v) and high current (1000a) are used. Surface discontinuities which show up after the establishment of a field may be photographed or videotaped to obtain a permanent record. Magnetic particle inspection provides an easy and accurate means of detecting surface and limited subsurface faults. With the technology in use a crack detection rate of 0.99 can be expected. A major drawback of the technique in harbor structures is that it is applicable only to steel or other ferromagnetic materials.

3. Eddy Current and Surface Potential Testing

Eddy current testing also provides a method of detecting cracks and other surface or near surface flaws such as pits and voids. This type of testing can be used on non-ferromagnetic materials which have a conductive surface, such as reinforced concrete. It is infrequently used underwater because of the extreme sensitivity of the probe to stand off distance, surface irregularity and surface condition.

Surface potential measurement, particularly when combined with automatic logging provides a method of assessing the corrosion potentials on the surface of a

reinforced concrete surface. With systems now available for underwater use, voltages in the range of 2v can be measured with a resolution of 1mv.

E. Ultrasonics

Ultrasonic determination of material thickness and location of subsurface discontinuities and flaws is the most commonly used underwater NDT technique. The technique is used routinely on steel and concrete structures with excellent results. Use to date with wood shows less accurate and reliable results, but efforts are underway to improve these. The best systems require the diver to handle only a small probe which transmits the reading to surface logging equipment and allows him to have one hand free for maintaining his position. To assist in relocating flaws for confirmation systems are available which incorporate a means by which the diver can see the signal generated topside or receive an audible tone. Difficulties encountered on steel surfaces with extensive corrosion giving false reading have been overcome by focusing .

The most comprehensive and most satisfactory programs of underwater NDT tailor the techniques to the structure being inspected and the results desired. Often it is desirable to use more than one technique and integrate the output to obtain the desired results. When more than one technique is indicated by the inspection goals it is false economy not to use them. The rapid changes and application of new technology to inspection of underwater structures by NDT makes it mandatory that Port Authorities keep themselves informed or seek informed advice when embarking on or continuing an inspectionprogram.

VI. HOW DO WE GET OUR TOOL TO THE JOB?

In conducting inspections in harbors and at port facilities there are two basic choices of inspection technology deployment: divers and remote operated vehicles (ROV's). Each has advantages for particular work and limitations which render it unsuitable for other work. Neither is a panacea. The method of deploying the inspection technology should be as carefully chosen as the inspection technology itself.

A. Divers

Divers are the traditional method of making underwater inspections. In the past when divers made only visual and touch inspections, engineers and management were dependent upon what the diver said to do their analyses

and draw conclusions. This is no longer the case. The diver has become a vehicle for taking the inspection technology to the work site and using it under the direction of topside personnel. The diver is the preferred inspection technology deployment system where:

* Water depth is relatively shallow so the diver has sufficient working time without building up a large decompression debt.

* The diver must maneuver in confined areas such as inside a piled structure.

* A high degree of manual dexterity is called for.

The diver should always be provided with voice communications with his tender. Additionally there should be means for real time data transmission to the surface when appropriate.

B. Remote Operated Vehicles

ROV's are used in inspection work at depths where divers do not have adequate working time and where a high degree of manual dexterity is not required. ROV's have a major advantage in that they do not expose a man to the hazards and limitations of being in the water. The vehicles are particularly suited for large area or linear surveys such as anchorage surveys, pipeline or cable surveys or tube crossing surveys where video inspection is the main interest. They are not well suited to work in and under piled structures where their umbilicals may become entangled in structure or debris and freeing them may be difficult. ROV's may be adapted to carry various inspection equipment, but a skilled operator is needed for precision placement.

VII. HOW ABOUT A PERMANENT RECORD?

There is little point in making the inspection if a record is not kept. It must be in a form that it can be used for engineering analysis, provide an input for management decisions and be understood by the new people. Several points made relative to record preparation are repeated here for emphasis:

* Data accumulation should be done topside,

* Records should be made in real time,

* Written records should be in a standard format, using standard terminology agreed on before the inspection,

* Photographs, videotapes and NDT records should be marked with the structure and location, the time and date and the identification of the recorder,

* Videotapes should be narrated.

The exact format and content of the inspection report is the perogative of the organization requiring the inspection. The Port Authority, who knows what he wants, and the underwater contractor, who knows what he can do, should work together to develop the report formats.

VIII. WHAT DOES ALL THIS MEAN?

The technology for making meaningful inspections on underwater structures exists now and is growing and changing rapidly. The existence and potential of this technology gives to those who are concerned with maintaining port facilities the means to plan maintenance programs more efficiently and stretch maintenance dollars further.